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18.1 INTRODUCTION

This chapter considers the following types of concrete structures:

1. Flat Slab
2. Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison of the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures. Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet (1.2 meters); formwork removal requires this clearance. When spans exceed 35 feet (10 meters), freeboard shall be increased to 5 feet (1.5 meters) above normal water.

Continuous slab spans are to be designed using pier caps or continuous transverse support for future superstructure replacement.

18.2 DESIGN SPECIFICATIONS AND DATA

(1) Specifications

Reference may be made to the design-related material as presented in the stated section of the following specifications:

State of Wisconsin, Department of Transportation
Standard Specification for Road and Bridge Construction
Section 502 - Concrete Bridges
Section 505 - Steel Reinforcement

American Association of State Highway and Transportation Officials (AASHTO)

(2) Allowable Stresses

The allowable stresses for concrete slab structures are based on Strength Design as follows:

| f'c = 4 ksi (28 MPa), specified strength based on a 28-day cylinder test for slabs.
| f'c = 3.5 ksi (24 MPa), all other concrete masonry.
| f_y = 60 ksi (420 MPa), specified yield strength based on Grade 60.
| n = ratio of modulus of elasticity of steel to concrete.
$= E_s / E_c$

(3) Structure Selection

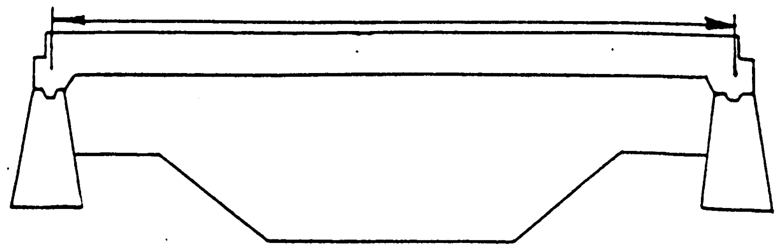
Prepare a preliminary plan showing the type of structure, span lengths, approximate slab depth, roadway width, live loading, etc. All concrete slab structures are limited to a maximum skew of 30 degrees.* The selection of the type of concrete slab structure is a function of the total span length required. Recommended span length ranges are shown for single- and multiple-span arrangements in Figure 18.1.

| Currently, voided slabs are not allowed. Some of the existing voided slabs have displayed excessive longitudinal cracking over the voids in the negative zone. This may have been caused by the voids deforming or floating-up due to lateral pressure during the concrete pour. Recent research indicates decks with steel void-formers have large crack widths above the voids due to higher stress concentrations.

If the positive span moments are held equal, the interior and exterior slab depths will be equal provided fatigue or crack control does not govern. Optimum span ratios are independent of applied MS-loading. For the following optimum span ratio equations based on strength controlling, L_1 equals the exterior span lengths and L_2 equals the interior span length or lengths for three or more span structures.

* Concrete slab structures with skews in excess of 30 degrees require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bridge Office.

Span < 50 feet (15 m) Use Flat Slab

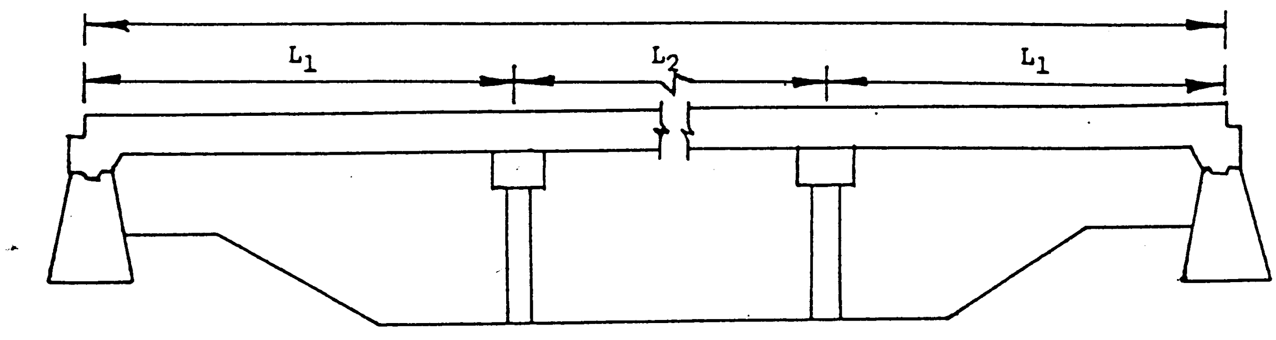


SINGLE SPAN

All Spans < 35 feet (11 m) ± Use Flat Slab Throughout

| Any Span ≥ 45 feet (14 m) ± < 70 feet (21 m) Use Haunched Slab Throughout

| Other Spans - Consider economics, aesthetics and clearances



TWO OR MORE SPANS

FIGURE 18.1

(4) Span Ratios

For flat slabs the optimum span ratio is obtained when $L_2 = 1.25L_1$. The optimum ratio for a three-span haunched slab results when $L_2 = L_1(1.43 - 0.002L_1)$ and for a four-span when $L_2 = 1.39L_1$.

Approximate slab depths for multiple-span flat and haunched slabs can be obtained from the graphs in Figure 18.2. The values are to be used for dead load computations and preliminary computations only and the final slab depth is to be determined by the designer.

18.3 DESIGN APPROACH

(1) Strength Procedure

AASHTO "Reinforced Concrete" Strength Design is employed in the design of concrete slab structures. Strength Design is also referred to as Load Factor Design, since the design loads (Service Loads) are multiplied by the appropriate load factors. The design is also modified by using capacity reduction factors which will be presented later in this section.

A. Stress-Strain Relationship

Stress is assumed proportional to strain in Service Design (working stress) below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangle, trapezoid, or parabola to relate the concrete compressive stress distribution to the concrete strain. Strength predictions are in agreement with comprehensive strength test results. Reference is made to Whitney's Ultimate Strength Analysis[1], Load Factor Design[2], and Strength Requirements[3], for the general procedure presented in this chapter. The rectangular uniform compressive stress block is used to determine the required tensile reinforcement. The representation of this assumption, a balanced section at ultimate strength, is shown in Figure 18.3.

(s) Span Length feet	meters	Slab Depth inches Haunched *	millimeters	Flat **	
20	6	---		12	300
25	7.5	---		14	350
30	9	---		16	400
35	10.5	---		18	450
40	12	---		20	500
45	13.5	16	400	22	550
50	15	17	425	24	600
55	16.5	18	450	26	650
60	18	20	500	---	
65	19.5	22	550	---	

* Note: These estimated slab depths @ mid-span apply to interior spans of three or more span structures with an end span length of approximately 0.7 times the interior span. Depths are based on dead load and live load deflection limits. Haunch, $(L) = .167L_2, d_s / D = .6$, were used. L_2 equals interior span length, (d_s) equals slab depth in span and (D) equals slab depth at haunch. These values include 1/2 inch (15 mm) wearing surface.

** Note: These values represent AASHTO's recommended minimum depths for continuous-spans (s+10)/30. For simple-spans add 10% greater depth and check criteria in Section 18.4. These values include 1/2 inch (15 mm) wearing surface.

The minimum slab depth is 12 inches (300 mm). Use increments of 1/2 inch (15 mm) to select depths > 12 inches.

FIGURE 18.2

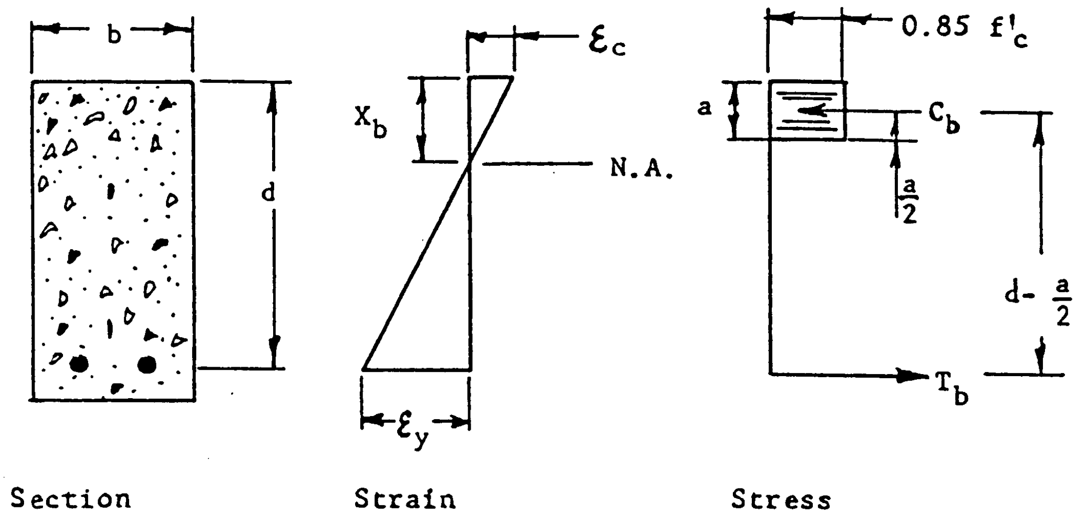


FIGURE 18.3

The balanced steel percentage P_b is computed from the following strain conditions:

1. The maximum strain at the extreme concrete compression fiber is 0.003.

2. Strain in the reinforcing steel and concrete are assumed directly proportional to the distance from the neutral axis.

3. Stress in reinforcement below the specified yield strength, f_y, for grade of steel used is taken as E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement is considered independent of strain and equal to f_y.

For a rectangular section, the compatibility of strains is proportional to the distance from the neutral axis and is expressed as:

$$\frac{\varepsilon_c}{\varepsilon_y} = \frac{X_b}{d - x_b} \qquad X_b = \frac{0.003d}{0.003 + \varepsilon_y}$$

if $E_s = 29{,}000{,}000$

$$X_b = \frac{87,000(d)}{87,000 + f_y}$$

Balanced conditions exist at a cross-section when the tension reinforcement reaches its specified yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain, 0.003.

Standard procedure is to limit the maximum percentage of tension reinforcement to a fraction of the steel area required for a balanced condition. This insures a ductile, under-reinforced condition and is expressed by the formula $P\max = 0.75P_b$ for rectangular sections with tension reinforcement only.

The balanced reinforcement ratio, P_b, is obtained by equating the internal compressive force, C_b, to the internal tensile force, T_b. Referring to Figure 18.3, the internal force equations are:

$$C_b = 0.85(f'c)(b)(a) = 0.85(f'c)(b)(B_1 X_b)$$

$$T_b = (A_{sb})(f_y) = (P_b)(b)(d)(f_y)$$

By equating $C_b to T_b$ and substituting for X_b, the balanced reinforcement ratio is:

$$P_b = \frac{(0.85)(B_1)(f'c)}{f_y} \qquad \frac{(87,000)}{(87,000 + f_y)}$$

The fraction B_1 is used as 0.85 for strengths of f'c up to 4 ksi (28 MPa) and is reduced continuously at a ratio of 0.05 for each 1000 psi (7 MPa) of strength in excess of 4000 psi.

For rectangular sections and flanged sections in which the compression flange thickness is equal to or greater than the compressive stress block depth, the design moment strength (tension reinforcement only) equals:

$$\phi Mn = \phi[A_s(f_y)(d)(1 - 0.6(P)\frac{f_y)}{f'c}]$$

$$= \phi[A_s(f_y)(d - \frac{a)}{2}]$$

Where

$$a = \frac{A_s f_y}{0.85(f'c)(b)}$$

ϕ is called the capacity reduction factor, which is used to reduce the computed theoretical strength of a structural element. This provides for the possibility that small variations in material strengths, workmanship, and dimensions may combine to result in undercapacity. The following values of ϕ are recommended:

For flexure .... $\phi = 0.90$
For shear .... $\phi = 0.85$
For bearing on concrete .... $\phi = 0.70$

For rectangular sections with compression reinforcement, the design moment strength equations are given in AASHTO - Section 8 "Concrete Design".

B. Load Factors

In Service Design, design loads are equal to the actual service loads to which the structure is subjected. It is a well-known fact that dead loads are more accurately determined than live loads. Strength Design recognizes that it is unreasonable to apply the same factor of safety to both loading conditions. Therefore, Strength Design requires higher overload factors applied to live loading. A typical equation for ultimate strength loading is:

Group 1 = 1.30 [D + 5/3 (L + I)] where D equals the dead load, L equals the live load, and I equals the percentage increase in live load due to impact.

In summary, Strength Design procedures more accurately predict conditions of actual material and structural behavior than Service Design procedures.

(2) Service Procedure

Service Design (working stress) procedure is used in applying AASHTO fatigue criteria. AASHTO Specifications place limits on reinforcement stress due to repeated applications of live loads. Fatigue criteria is required due to the high concrete and reinforcement stresses resulting from load factor design and the high allowable stresses for Grade 60 reinforcement in Strength Design.

AASHTO specifications consider fatigue stress limits for steel reinforcement by employing the effects of stress range. The range between the maximum and minimum stress in straight reinforcement caused by live load plus impact at service load shall meet AASHTO specifications. Reinforcement fatigue strength is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

Reinforcing bars in bridge superstructures are more likely to be stressed near the critical fatigue stresses than is the surrounding concrete. For this reason, particular care must be taken to check all potential fatigue locations. Longitudinal bars in all types of bridges are checked for fatigue at locations of maximum service load stress range and at bar cutoffs. In regions where stress reversal takes place, continuous concrete slabs are doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered. For fatigue limits, only the
elastic effects of service load needs to be taken into account. Therefore, a modular ratio of n = E_s/E_c is used to transform the compression reinforcement for fatigue stress
computations.

Current AASHTO specifications require reinforcement for shrinkage and temperature stresses near exposed surfaces of slabs not otherwise reinforced. The total area of reinforcement provided must be at least 1/8 sq. in./foot (265 sq. mm per meter) and be spaced not further apart than three times the slab thickness or 18 inches (450 mm).

(3) Distribution of Flexural Reinforcement

The use of high-strength steels and the acceptance of Strength Design concepts where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over bars, and area of concrete in the zone of maximum tension surrounding each individual reinforcing bar.

AASHTO specifications requires that where the yield strength of the reinforcement exceeds 40 ksi (300 MPa), the detailing of bars should be such that the tensile stress (f_s) in the reinforcement at service loads does not exceed $Z \div (d_c A)^{1/3}$, but f_s shall not be greater than $0.6 f_y$. Refer to AASHTO specifications for notation. The value of Z shall not exceed 130 (23,000 N/mm) for severe exposure (top reinforcement), or 170 (30,000 N/mm) for moderate exposure (bottom reinforcement). When checking crack control for top slab reinforcement, deduct the 1/2 inch (15 mm) wearing surface. The allowable stress will increase by about 10 to 15 percent. For application, refer to design example.

(4) Design Procedure

A. Dead Load

A trial slab depth is obtained by referring to Figure 18.2. Slab dead load is computed by using a concrete weight with no adjustment in weight for bar steel reinforcement.

AASHTO specifications allow the weight of curbs, parapets, medians, railings, sidewalks, and other dead loads placed after the slab has cured to be equally distributed across the width of the slab. However, AASHTO specifies the provision of longitudinal edge beams on concrete slab structures with main steel parallel to the direction of traffic.

For WisDOT, standard procedure is to provide for edge beams in the positive zones only. This assumption is based on the location of the live loads producing the maximum moments. In the positive zone, the live load is adjacent or near the point of maximum positive moment location on the edge beam; distribution of live load to the edge of the slab is prevented. In the negative zone, the live loading is not necessarily adjacent to the point of maximum negative moment and a better live load distribution across the width of the slab is obtained.

A post-dead load of 20#/ft^2 (1.0 kN/m^2) is to be included in all designs in order to accommodate a possible future wearing surface.

B. Live Load Distribution

This criteria is presented in AASHTO "Distribution of Loads and Design of Concrete Slabs". For concrete slab structures with the main reinforcement perpendicular to traffic, the live load moment is obtained from AASHTO formulas or tables.

These values may be used without computing the distribution of wheel loads. However, for concrete slab structures with main reinforcement parallel to traffic, the wheel load (1/2 lane) is distributed over a width to be determined.

C. Live Load and Impact

The impact formula is given in AASHTO. It is applicable to all concrete slab superstructures. The live load moment and shears are obtained from computer programs.

NOTE: Concrete slab structures are required to have the capacity to carry the Standard Permit Vehicle (as shown in Chapter 45) having a minimum gross load of 190 Kip (845 kN) while also carrying future wearing surface loads. The distribution factor specified in Chapter 45 is used for this loading so it will probably not govern the slab design.

D. Slab Design

Based on the trial slab depth and main reinforcement parallel to traffic, the Continuous Beam Analysis Program outputs the area of steel, bar size and spacing. The area of reinforcing steel required is controlled by strength, fatigue, or crack control. The designer may wish to try different slab depths to determine the optimum cost of different combinations of reinforcement and concrete.

Generally, shear does not control in determining the required slab depth. Slabs designed according to AASHTO 3.24.3 are considered satisfactory in shear.

Total slab depth is equal to d, plus one-half a bar diameter and the concrete
| cover. The concrete cover on the top bars is 2 1/2" (65 mm) which includes a
| 1/2" (15 mm) wearing surface; the bottom bar cover is 1 1/2" (40 mm).

For multiple concrete slab structures, if the slab depths for adjacent spans are within 1" (25 mm), the slab depths for all spans are made equal to the maximum slab depth. The haunch depth for haunched slabs is proportioned on the basis of total slab depth (ds) outside the haunch. Total haunch depth $D = d_s / 0.6$; this is a starting approximation. If total slab depths differ by more than 1" (25 mm), the trial haunch depth is computed from the total slab depth of Span 2.

NOTE: For a tapered haunch see Standard 18.1 for relative slab and haunch depths.

An economical haunch length is approximately equal to from $0.15L_2$ to $0.18L_2$ where L_2 is the length of Span 2.

E. Longitudinal Slab Reinforcing Steel

The minimum clear spacing between adjacent bars for longitudinal slab steel
| is 3 1/2" (90 mm). This spacing facilitates compliance with AASHTO specifications for distribution of tension reinforcement to reduce flexural cracking when Load Factor Design is employed. Bars can be bundled if the calculated stress in the reinforcement from service loads does not exceed the stress allowed by the crack control criteria. Bundled bars are taken as one bar for computing the effective tension area of concrete.

Negative reinforcement is terminated at two points. One-half of the bar steel is cut off where half of the area of steel can carry the remaining moment unless fatigue or crack control governs at that point or the distance from the support is less than the development length of the bar. The bars are extended beyond this cutoff point for a distance equal to the effective depth of slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. The variable slab thickness is considered for computing moment capacity between the tenth points on haunched slabs. The remaining bars are terminated where the moment envelope equals zero. At least one-third of the total bar steel must extend beyond this location (point of inflection) not less than the effective depth of the member, 12 bar diameters, or 1/16 of the clear span, whichever is greater.

One-half of the positive reinforcement can be terminated at a distance equal to or greater than the development length from the point of maximum positive moment where half of the bar steel can carry the remaining moment unless fatigue or crack control governs. The reinforcement is extended beyond this point for a distance equal to the effective depth of slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. At least one-third of the positive moment reinforcement in simple slabs and one-fourth of the positive moment reinforcement in continuous slabs is extended along the same face of the slab into the support.

F. Transverse Distribution Reinforcement

Distribution reinforcement is placed in the bottom of all concrete slabs to provide for lateral distribution of concentrated loads. The specification is referred to in AASHTO Section 3 - Article 3.24.10.2. This states that the amount of steel is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

| Percentage = $100/(s)^{1/2}$ Maximum 50% where S equals the effective
| span of the slab in feet.

The distribution reinforcement is to be placed in the middle half of the slab span. In the outer quarters of the span, not less than 50 percent of the above amounts is to be used. The above formula is very conservative when applied to slab structures since this specification was primarily drafted for the relatively thin slabs on stringers.

Refer to Standards 18.1 and 18.2 for remaining longitudinal and transverse shrinkage and temperature bar steel requirements.

G. Edge Beam Design

The following procedure is used to design edge beams for concrete slab structures as required by AASHTO Section 3, Article 3.24.8 and applied to the positive moment zone only. The reinforcement in the edge beam section is to always equal or be greater than that required by the slab in the positive zone. The portion of the slab which is overlapped by the curb or sidewalk is considered as effective as part of the edge beam and is designed to carry its own dead load, plus the dead load of the curb or sidewalk and railing dead load, whichever may be the case. For the standard concrete safety parapet or median barrier, the slab width is increased to the width overlapped plus one-half of the total slab depth, for dead load calculations.

The section formed by the overlapping portions of the slab and curb, sidewalk, safety parapet, or median barrier is designed to carry sidewalk live load, and 0.2* (pos. lane load mom). This is used in lieu of 0.1PS for live load moment since it is readily available from the Continuous Beam Analysis Computer Program. If a thickened section is created by the curb or sidewalk, the entire depth of section is used as the edge beam depth. If the edge of slab is overlapped by a safety parapet or a median barrier, the depth of section is taken as the depth of slab plus 12 inches (300 mm). This is a conservative approximation used in place of a more detailed analysis for an unsymmetrical section. Reference may be made to the design example in Section 18.5 of this chapter for edge beam requirements. Deflection joints in the safety parapet and median barrier are to be centered on either side of the point of maximum positive moment in the slab span by the designer. No edge beam action is considered in the negative moment zone; all railing, curb, sidewalk, safety parapet, and median barrier dead loads and sidewalk live load are distributed equally across the roadway slab.

H. Bar Steel Splice

All bar steel splices are to be staggered, if possible, and located by the Design Engineer. Lap splices of bundled bars are based on the lap splice length required for individual bars of the same size as the bars spliced.

I. Transverse Reinforcement at Piers

If the concrete superstructure rests on a pier cap or directly on columns, additional transverse reinforcement is required. A portion of the slab above the pier is designed as a continuous pier beam along the centerline of the superstructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a haunched slab, the width of the transverse slab member is usually equal to one-half the center to center spacing between columns for the positive moment zone. The width equals the diameter of the column plus 6 inches (150 mm) for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Use a width in the positive moment zone to satisfy the shear strength criteria without using stirrups. Reference may be made to the design example in Section 18.5 of this chapter for the computations relating to transverse reinforcement.

18.4 DESIGN CONSIDERATIONS

(1) Camber and Deflection

All concrete slab structures shall be designed to meet live load deflection and camber limits. Live load deflections for concrete slab structures are limited to L/1200 based on full structure width and gross moment of inertia.

Dead load deflections for concrete slab structures are computed using the gross moment of inertia. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Full camber is based on multiplying the dead load deflection values by a factor of <u>three</u>. Most of the excess camber is dissipated during the first year of service which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement.

A. Simple-Span Concrete Slabs

Camber for simple-span slabs is limited to 2 1/2 inches (65 mm). * For simple-span slabs, Wisconsin Office practice indicates that using a minimum slab depth of 1.1* (S + 10)/30, and meeting the live load deflection and camber limits stated in this section, provides an adequate slab section for most cases.

B. Continuous-Span Concrete Slabs

Maximum allowable camber for continuous-span slabs is 1 3/4 inches (50 mm).*

*If full camber is exceeded, the designer is to redesign the concrete slab depth to meet the criteria. Dead load deflection based on I-gross shall not exceed one-third full camber.

(2) Deflection Joints & Construction Joints

The designer should locate deflection joints for concrete slab structures according to Standard 18.1.

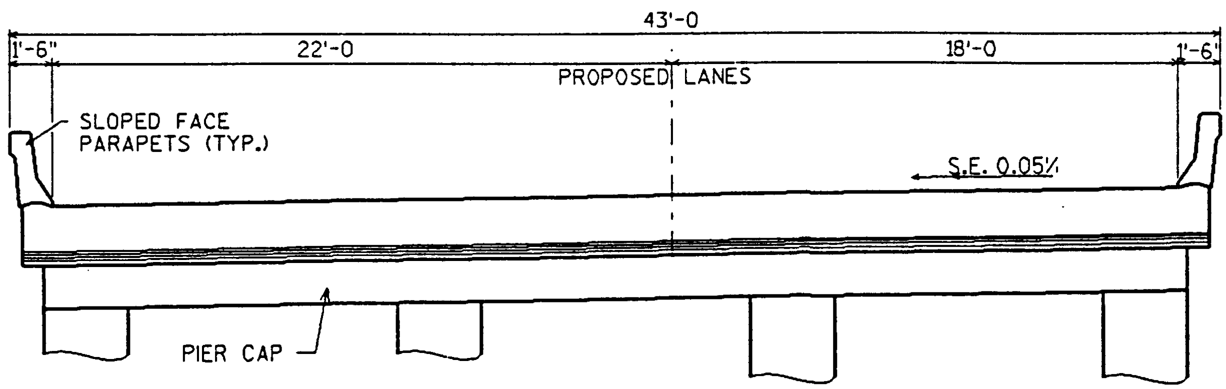
Refer to Bridge Manual, Chapter 17, for recommended construction joint guidelines.

18.5 DESIGN EXAMPLE (Load Factor Design Method).

NOTE: The following example uses English units.

A continuous haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The AASHTO specifications are followed as stated in the text of this chapter. Design a 1.0 (ft) wide strip of slab.

Structure Preliminary Data



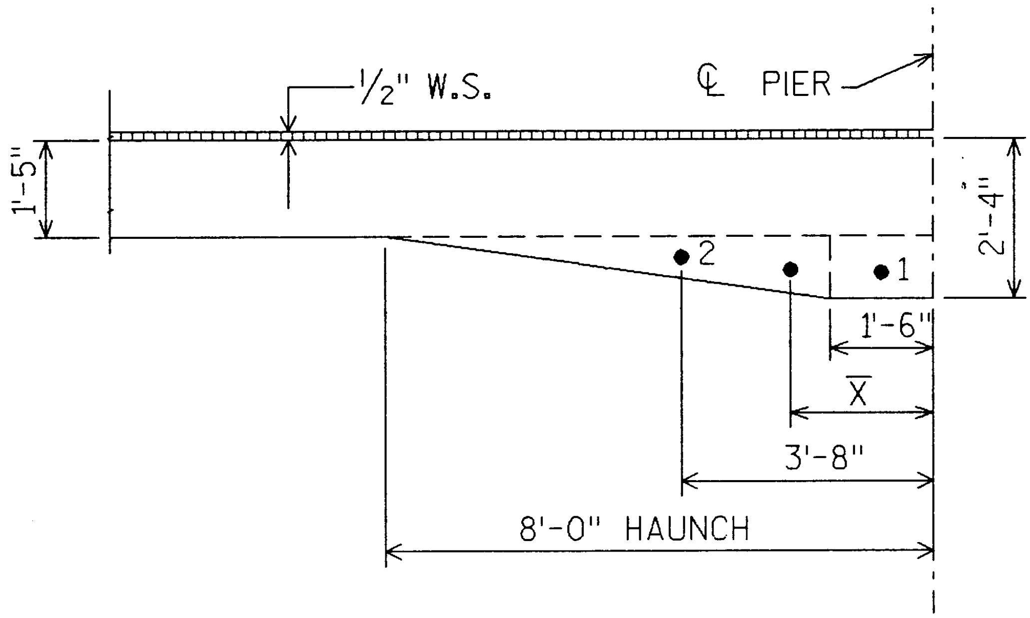
Non-Tapered Haunch is used in this example.
Span Lengths: 38'-0, 51'-0, 38'-0.
Live Load: HS20
Skew 6° 00' RHF.
(A-1) Abutments at both ends.
Parapets placed after Falsework is released.
Concrete (Slab): f'c = 4,000 p.s.i.
Reinforcement: fy = 60,000 p.s.i.
Concrete Wt. = 150 #/ft^3.
Parapet Wt. = 338 #/ft.

Dead Loads

Refer to Figure 18.2, for a span length of 51 feet. The slab depth is estimated at 17 inches (not incl. ½" W.S.). The haunch depth (D) is approximately equal to d_s divided by 0.6 where d_s is the slab depth.

D = 17/0.6 = 28 in. (not incl. ½" W.S.)

The length of haunch is approximately 0.15 to 0.18L_2 or say 8'-0 which equals 0.157L_2.



Note: For a Tapered Haunch see Standard 18.1 for relative slab and haunch depths.

For hand computations determine partial haunch dead load. Determine value of $\overline{X}$ and distribute haunch weight uniformly over twice this distance. Haunch dead load is computed automatically for Bridge Office personnel by their computer program.

Slab Dead Load = (17/12)(1)(150) = 213 #/ft (on a 1'-0 width)

Rail, curb, and parapet dead load is distributed over the full width of the slab for negative moment only. In the positive moment area these loads are not distributed and are placed on an edge beam.

$$\text{Parapet Dead Load} = \frac{(2)(338)\#}{42.17'} = 16\#/ft\ (on\ a\ 1'\text{-}0\ width)$$

A post dead load of 20 #/ft^2, for possible future wearing surface (F.W.S.), plus the ½ inch wearing surface load (6 #/ft^2) must also be included in the design of the slab.

Check adequacy of chosen slab thickness by looking at live load def'l. and camber limits.

Live Load Deflection Check

Allowable deflection = L/1200 (based on Ig and def'l. of deck as a unit).

Span 1: L/1200 = 0.38" > 0.314" (actual LL defl.) O.K.

Span 2: L/1200 = 0.51" = 0.510" (actual LL defl.) O.K.

Camber Check

Max. DL defl. (based on Ig) at C/L Span 2 = 0.250".

| Therefore, max. camber = (3)(0.250") = 0.75" < 1 ¾" O.K.

Live Load Distribution (AASHTO 3.24.3.2)

Wheel loads are distributed over a width, E = 4.0 + 0.06(S) ≤ 7'-0 where S equals the effective span length in feet. Lane loading is distributed over a width of 2E.

The distribution factor, DF, is computed for a unit width of slab equal to one foot. The distribution factor is:

$$DF = \frac{1}{E}$$

For spans 1 & 3:

E = 4.0 + 0.06(38) = 6.28'

DF = $\frac{1}{6.28} = 0.159$

For span 2:

E = 4.0 + 0.06(51) = 7.06' must be ≤ 7.0'

DF = $\frac{1}{7.0} = 0.143$

NOTE: Concrete Slab Structures are to be designed to also have the capacity to carry the Standard Permit Vehicle (as shown in Chapter 45) having a gross load of 190 kips while also carrying future wearing surface. (This example does not include this).

Service Load Moments (ft-k) (on a 1'-0 width)

Point	DLM ▲	Curb DL*	+(L+I)	-(L+I)	Curb DL+ DLM+(L+I)	DLM-(L+I)
0.1	8.9	1.5	16.5	- 3.1	25.4	7.3
0.2	14.7	2.4	26.7	- 6.2	41.4	10.9
0.3	17.3	2.8	31.5	- 9.3	48.8	10.8
0.4	16.7	2.7	32.5	-12.4	49.2	7.0
0.5	13.0	2.1	31.0	-15.5	44.0	- 0.4
0.6	6.1	1.0	28.1	-18.6	34.2	-11.0
0.7	- 3.9	- .7	21.4	-21.7	17.5	-26.3
0.8	-17.1	-2.8	11.8	-24.9	- 5.3	-44.8
0.9	-33.7	-5.5	6.4	-28.0	-27.3	-67.2
1.0	-54.8	-8.7	8.4	-35.1	-46.4	-98.6
1.1	-27.5	-4.5	5.9	-21.8	-21.6	-53.8
1.2	- 7.5	-1.2	12.1	-18.1	4.6	-26.8
1.3	6.7	1.1	22.5	-14.7	29.2	- 6.9
1.4	15.2	2.5	28.9	-11.3	44.1	6.4
1.5	18.1	3.0	30.3	- 9.9	48.4	11.2

* Values in this column incl. F.W.S. & parapet dead load distributed across width of deck, and would apply to negative moment regions. In positive moment regions, away from edge beam, the Curb DL moment would be less than this because parapet DL is not distributed there. For simplicity we will use values in above table without modification in positive moment region.

▲ Dead load moment includes ½" W.S. and slab dead load.

Longitudinal Slab Reinforcement (on a 1'-0 width)

Positive moment reinforcement for span 1.

Design for Strength At the 0.4 point of span 1,
$(+M_u)$ = 1.3(DLM + Curb DL + 5/3 (L+I)) = 1.3(16.7 + 2.7 + 5/3(32.5))
= 95.6 ft-k.

b = 12" (for a 1'-0 design width)

d = 17.5" – (1.5 + 0.6 + .5) = 14.9"

Solve for steel area using Table (R_n vs. ρ)

$$R_n = \frac{Mu}{\phi bd^2} = \frac{(95.6)(12)(1000)}{(0.9)(12)(14.9)^2} = 478.5\,psi$$

$$\rho = 0.0086$$

$$A_s = \rho(b)(d) = (.0086)(12)(14.9) = 1.54\,in^2/ft.$$

Check for Fatigue (AASHTO 8.16.8.3)

Steel: Check fatigue by AASHTO formula for max. stress range on steel. At 0.4 point of span 1 the moment range is [+(L+I)] – [-(L+I)] because moment range stays in tensile zone = (+32.5)-(-12.4) = 44.9 ft-k.

$$Allowable\ f_f = 21. - .33 f_{min} + 8(.3) = 22.1\ ksi$$

$$Where .33 f_{min} = .33 \frac{[(DLM + Curb\, DL - (L+I))x12)]}{(A_s)(j)(d)}$$

$$= 0.33 \frac{[(16.7 + 2.7 - 12.4)x12)]}{(1.54)(.9)(14.9)} = 1.34\, ksi$$

$$A_s\ (\text{min.}) = \frac{M(range)}{f_f\, j\, d} = \frac{(44.9)(12)}{(22.1)(0.9)(14.9)} = 1.82\ in^2/ft\ \underline{\text{controls}}$$

Concrete: Fatigue check using max. allowable concrete stress of 0.5 f'c @ 0.4 point of span 1.

$$f_c = \frac{2M(range)}{k\, j\, bd^2} = \frac{(2)(44.9)(12)}{(0.30)(0.9)(12)(14.9)^2} = 1.50\ ksi$$

less than allowable stress of (2.0 ksi. = 0.5 f'c).

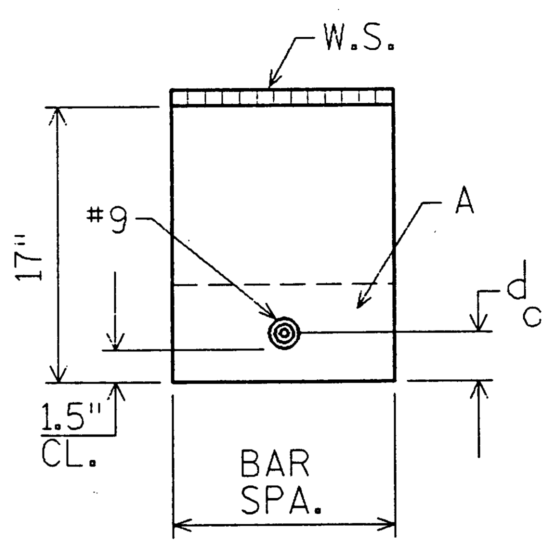
Check Crack Control (0.4 pt.) (AASHTO 8.16.8.4)

Crack control is governed by the equation:

$$f_s = \frac{Z}{(dc\ A)^{1/3}}\ \text{not to exceed } 0.6\ f_y.$$
allow.

Z = 170 k/in for bottom steel reinf.

A_s required is 1.82 in^2/ft (from page 21).



Try: #9's at 6 ½" c-c spacing
(A_s = 1.85 in^2/ft)

d_c = clr. Cover + ϕ bar/2 = 1.5 + (1.128)/2 = 2.064 in.

A = (2)(dc)(bar spa.) = (2)(2.064)(6.5) = 26.83 in^2

$f_s = \dfrac{170}{[(2.064)(26.83)]^{1/3}} = 44.6\ ksi$ > (36 ksi = 0.6 f_y)

allow.

therefore f_s allow. = 36 ksi

$$f_s(act) = \frac{(DLM + Curb\,DL + (L+I)}{(A_s\ j\,d)} = \frac{(16.7+2.7+32.5)(12)}{(1.85)(.9)(14.9)}$$

= 25.10 ksi

f_s (act) is less than (36.0 ksi = f_s allow.), (O.K.)

Use: #9's @ 6 ½" c-c spacing in span 1. (Max. positive reinforcement).

Max. reinf. Check (AASHTO 8.16.3.1)

max. $\rho = .75\ (\rho_b) = 0.0214\,(not\,incl.\ \rho')$.

actual $((\rho) = As/(b)(d) = (1.85)/(12)(14.9) = 0.0103 < \max(\rho)(O.K.)$

Min. reinf. check (AASHTO 8.17.1)

$fr = 7.5\sqrt{f'c} = 7.5\sqrt{4{,}000} = 474\,psi$

$Mcr = fr\,(I)/(c), where\ I = \dfrac{1}{12}(b)(h)^3 = 4913\,in.^4, c = 8.5''$

$Mcr = (.474)(4913)/(8.5)(12) = 22.83\ ft-k.;1.2\,Mcr = 27.4\ ft.k.$

$\phi\,Mn = 112.7\ \ ft-k > 1.2\ Mcr\,(O.K.)$

Negative moment reinforcement at Piers

At C/L Pier 1:

Design for Strength

Max. ($-M_u$) = 1.3 (DLM + Curb DL + 5/3 (L+I)
= 1.3 (54.8 + 8.7 + 5/3 (35.1)) = 158.6 ft-k.
b = 12" (for a 1'-0 design width)
d = 28.5" – (2.5 + 0.6) = 25.4"

$$R_n = \frac{(158.6)(12)(1000)}{(0.9)(12)(25.4)^2} = 273.1\,psi,\ \rho = 0.0048$$

$$A_s = \rho(b)(d) = (0.0048)(12)(25.4) = \underline{1.46\ \text{in}^2/\text{ft}}$$

Check for Fatigue (C/L Pier) (AASHTO 8.16.8.3)

Steel: Fatigue check for a moment range of [+(L+I)] – [-(L+I)] , because moment range stays in tensile zone.
= (8.4) – (-35.1) = 43.5 ft-k.

$$f_f\,(allow.) = 21. - .33 f\min + 8(.3) = 16.8\ ksi$$

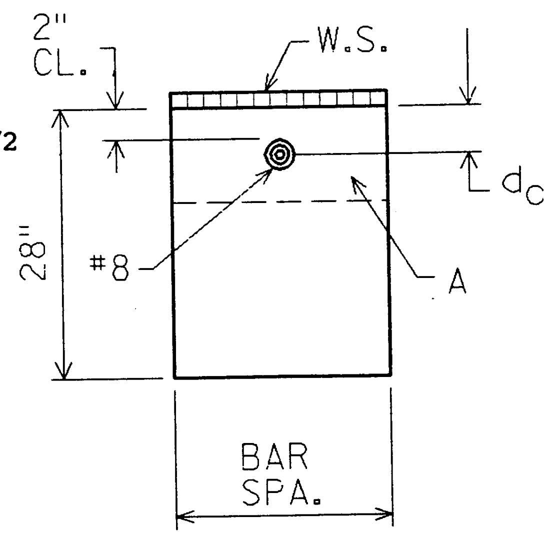
$$where\ .33 f\min = .33\ \frac{[(DLM + Curb\,DL - +(L+I))]x12}{(A_s)(j)(d)}$$

$$= .33\ \frac{(54.8+8.7-8.4)x12}{(1.46)(.9)(25.4)} = 6.6\ ksi$$

$$A_s(\min.) = \frac{M(range)}{f_f\, j\, d} = \frac{(43.5)(12)}{(16.8)(.9)(25.4)} = 1.36\, in^2/ft$$

Check Crack Control (C/L Pier) (AASHTO 8.16.8.4)

Check steel required for crack control. Try #8's @ 6" c-c spacing (A_s = 1.58 in^2/ft) > 1.46 in^2/ft.



Z = 130 k/in for top steel reinf.

d^*_c = clear cover - 1/2" wearing surface + ϕ bar/2
= 2 1/2" - 1/2" + 1/2" = 2 1/2"

A = (2)(2.5)(6.0) = 30.0 in^2.

$$f_s\,allow. = \frac{130}{[(2.500)\ (30.0)]^{1/3}} = 30.82\ ksi$$

$$f_s(act.) = \frac{(DLM + Curb\ DL + (L+I))}{(A_s\ j\ d)}$$

$$= \frac{(54.8 + 8.7 + 35.1)(12)}{(1.58)(.9)(25.5)}$$

= 32.63 ksi > f_s allow. (N.G.)

* For top slab steel, reduce d_c by wearing surface thickness.

Try: #8's at 5 ½" c-c spacing (A$_s$ = 1.72 in^2/ft).

Check this reinforcement vs. crack control criteria.

d_c = 2 ½" – ½" + 1.00"/2 = 2.50"

A = (2)(2.50)(5.5) = 27.50 in^2

$$f_s\ allow. = \frac{130}{[(2.50)\ (27.50)]^{1/3}} = 31.78\, ksi$$

$$f_s\ (act.) = \frac{(DLM + Curb\, DL + (L+I))}{(A_s)(j)\ (d)} = \frac{(54.8+8.7+35.1)(12)}{(1.72)\ (.9)\ (25.5)}$$

= 29.97 ksi < ($f_{s\ allow.}$ = 31.78 ksi) (O.K.)

Use: #8's at 5 ½" c-c spacing at pier. (Max. negative reinforcement)

Max. reinf. check (AASHTO 8.16.3.1)

actual (ρ) = As/(b) (d)=(1.72)/(12)(25.5)=.0056 < max (ρ) O.K.

Min. reinf. check (AASHTO 8.17.1)

$$fr = 7.5\sqrt{f'c} = 7.5\sqrt{4{,}000} = 474\ \ p.s.i.$$

$Mcr = fr(I)/(c),\ where\ I = 21{,}952\ in^4,\ c = 14"$

Mcr = (.474) (21,952)/(14) (12) = 61.9 ft.-k; 1.2 Mcr = 74.3 ft-k.
ϕMn = 187 ft-k > 1.2 Mcr O.K.

Positive moment reinforcement for span 2.

Design for Strength. At the 0.5 point of span 2,
(+Mu) = 1.3 (DLM + Curb DL + 5/3 (L+I)) = 1.3 (18.1 + 3.0 + 5/3 (30.3))
= 93.08 ft-k.
b = 12" (for a 1'-0 design width

d = 17.5" – (1.5 + 0.6 + 0.5) = 14.9

$$R_n = \frac{Mu}{\phi bd^2} = \frac{(93.08)(12)(1{,}000)}{(.9)(12)(14.9)^2} = 465.8\, psi$$

ρ = 0.0084
A$_s$ = ρ (b)(d) = (0.0084) (12) (14.9) = 1.50 in^2/ft.

Check for Fatigue

Steel: At 0.5 point of span 2 the moment range is [+(L+I)] - [-(L+I)] ,
Because moment range stays in tensile zone.
= (+30.3) – (-9.9) = 40.2 ft-k.

$$Allowable\ f_f = 21. - .33\, f\min. + 8(.3) = 21.2\ ksi$$

$$Where\ .33\, f\min = 0.33\ \frac{[(DLM + Curb\ DL - (L+I))x12]}{(A_s)\ (j)\ (d)}$$

$$= 0.33\ \frac{[(18.1) + 3.00 - 9.9)x12]}{(1.50)\ (.9)\ (14.9)} = 2.22\ ksi$$

$$A_s\ (\min) = \frac{M(range)}{f_f\ j\ d} = \frac{(40.2)(12)}{(21.2)(.9)(14.9)} = 1.70\ in^2/ft.$$ controls

Concrete: Fatigue check using max. allowable concrete stress of 0.5 f'c at 0.5 point of span 2.

$$f_c = \frac{2M(range)}{k\ j\ bd^2} = \frac{(2)(40.2)(12)}{(.30)(.9)(12)(14.9)^2} = 1.34\ ksi$$

less than allowable stress of (2.0 ksi = 0.5 f'c).

Check Crack Control (0.5 pt.)

Z = 170 ^{k}/in. for bottom steel reinf.

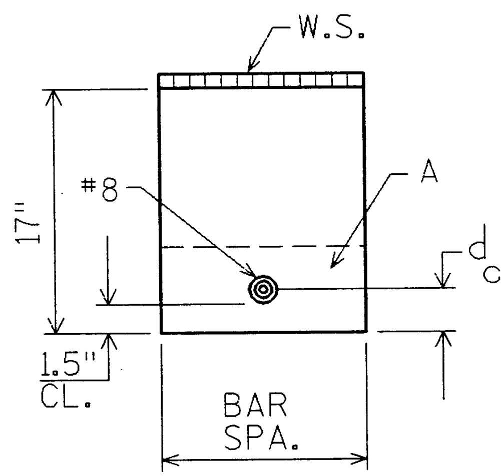
A_s required is 1.70 in^2/ft. (from page 25)

Try: #8's at 5 1/2" c-c spacing, (A_s = 1.72)

d_c = 1.5 + 1.000/2 = 2.000 in.

A = (2)(2.000)(5.5) = 22.0 in ^{2}.

$$f_s\ allow. = \frac{170}{[(2.00)(22.0)]^{1/3}} = 48.15\ ksi > 0.6\ fy$$



therefore f_s allow. = 36 ksi

$$f_s(act) = \frac{(DLM + Curb\ DL + (L+I))}{(A_s)\ (j)\ (d)} = \frac{(18.1 + 3.00 + 30.3)(12)}{(1.72)(.9)(15.0)} = 26.6\ ksi$$

f_s(act) is less than (36.0 ksi = $f_s$allow.), O.K.

Use #8's at 5 1/2" c-c spacing in span 2. (Max. positive reinforcement).

Max. reinf. check O.K.

Min. reinf. check O.K.

Negative moment reinforcement at the haunch-slab intercept.

Check strength at 0.789 Point of span 1.

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

Max. ($-M_u$) = 1.3 (DLM + Curb DL + 5/3(L+I)) = 1.3 (15.6 + 2.56 + 5/3 (24.5))
= 76.69 ft-k.

b = 12" (for a 1'-0 design width)

d = 17.5" – (2.5 + 0.5) = 14.5"

$$R_n = \frac{(Mu)}{\phi b d^2} = \frac{(76.69)(12)(1{,}000)}{(.9)(12)(14.5)^2} = 405.3 \ psi$$

$$\rho = 0.0072$$

A_s = (.0072)(12)(14.5) = 1.25 in^2/ft. < A_s (prov'd) = (1.72 in^2/ft.) O.K.

Check for Fatigue (.789 Pt.) of span 1

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

Steel: At 0.789 point of span 1 the moment range is [+(L+I)] – [-(L+I)], because moment range stays in tensile zone.
= (+12.86) – (-24.5) = 37.36 ft-k.

$$Allowable\ f_f = 21. - .33 f\min. + 8(.3) = 22.46\ ksi$$

$$Where\ .33\ f\min. = 0.33\ \frac{[(DLM + Curb\,DL - (L+I))\ x\ 12]}{(A_s)\ (j)\ (d)}$$

$$= \frac{[0.33\ (15.6 + 2.56 - 12.86)\,x12]}{(1.72)(.9)(14.5)} = 0.93\ ksi$$

$$A_s(\min) = \frac{M(range)}{f_f\ j\ d} = \frac{(37.36)(12)}{(22.46)(.9)(14.5)} = 1.53 in^2/ft. < A_s\ (prov'd)\ O.K.$$

Check Crack Control (.789 Pt.) of span 1

Z = 130^k/in. for top steel reinf.

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

d_c = 2 ½" – ½" + 1.000/2 = 2.50"

A = (2)(2.50")(5.5) = 27.50 in^2.

f_s allow = $\dfrac{130}{[(2.50)(27.50)]^{1/3}} = 31.78\ ksi$

f_s (act) = $\dfrac{(DLM + Curb\ DL + (L+I))}{(A_s)\ (j)\ (d)} = \dfrac{(15.6+2.56+24.5)(12)}{(1.72)(.9)(14.5)}$

= 22.80 ksi < ($f_s$allow.) O.K.

Max. reinf. check. O.K.

Min. reinf. check. O.K.

<u>Check strength at 0.157/0.843 Point of span 2.</u>

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

Max. (-Mu) = 1.3(DLM + Curb DL + 5/3 (L+I)) = 1.3(16.1 + 2.6 + 5/3 (19.7)) = 67.0 ft-k.

b = 12" (for a 1'-0 design width)

d = 17.5" – (2.5 + 0.5) = 14.5"

$$R_n = \frac{(Mu)}{\phi bd^2} = \frac{(67.0)(12)(1{,}000)}{(.9)(12)(14.5)^2} = 354\, psi$$

$\rho = 0.0063$

A_s = (.0063)(12)(14.5) = 1.10 in^2/ft. < A_s (prov'd) = 1.72 in^2/ft. O.K.

<u>Check for Fatigue (0.157/0.843) Pt. Of span 2</u>

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

<u>Steel:</u> At 0.157 point of span 2 the moment range is [+(L+I)] – [-(L+I)], because moment range stays in tensile zone.
= (+9.5) – (-19.7) = 29.2 ft-k.

$$Allowable\ f_f = 21. - .33\, f\,\mathrm{min.} + 8(.3) = 21.77\ ksi$$

$$Where\ .33\ f\,\mathrm{min.} = 0.33\,\frac{[(DLM + Curb\,DL - (L+I))\,x12}{(A_s)\ (j)\ (d)}$$

$$= \frac{0.33[(16.1+2.6-9.5)\,x12]}{(1.72)(.9)(14.5)} = 1.63\ ksi$$

$$A_s(\mathrm{min}) = \frac{M(range)}{f_f\ j\ d} = \frac{(29.2)(12))}{(21.77)(.9)(14.5)} = 1.23\ in^2/ft. < A_s\,(prov'd.)\ O.K.$$

Check Crack Control (0.157/0.843) Pt. of span 2

Z = 130^k /in. for top steel reinf.

Check #8 at 5 ½" c-c spacing (as req'd. at pier).

d_c = 2.50"

A = 27.50 in^2.

$$f_s allow. = \frac{130}{[(2.50)(27.50)]^{1/3}} = 31.78\ ksi$$

$$f_s\,(act).= \frac{(DLM + Curb\,DL + (L+I))}{(A_s)\,(j)\,(d)} = \frac{(16.1+2.6+19.7)(12)}{(1.72)(.9)(14.5)} = 20.52\ ksi < (f_s\ allow.)\ O.K.$$

Max. reinf. check. O.K.

Min. reinf. check. O.K.

Bar Steel Cutoffs

Span 1 Positive Moment Reinforcement. (Cutoffs)

Preliminary bar steel cutoff location for positive moment is determined when one-half the steel required at 0.4 pt. has the capacity to handle the ultimate moment at this location. However, the service load stresses must meet the fatigue and crack control requirements at the cutoff location. The factored moments (Mu) at the 0.1 points have been plotted on the following page. The capacities (ϕMn) of #9 at 6 ½" and #9 at 13" are also shown. b = 12" (for a 1'-0 design width).

Capacity of #9 at 6 ½" (A_s = 1.85 in^2/ft.) d = 14.9" (AASHTO 8.16.3.2)

C = (.85f'c)(b)(a) = (.85)(4.0)(12)(a) = 40.8(a)

T = A_s(fy)=(1.85)(60)=111^k , C = T

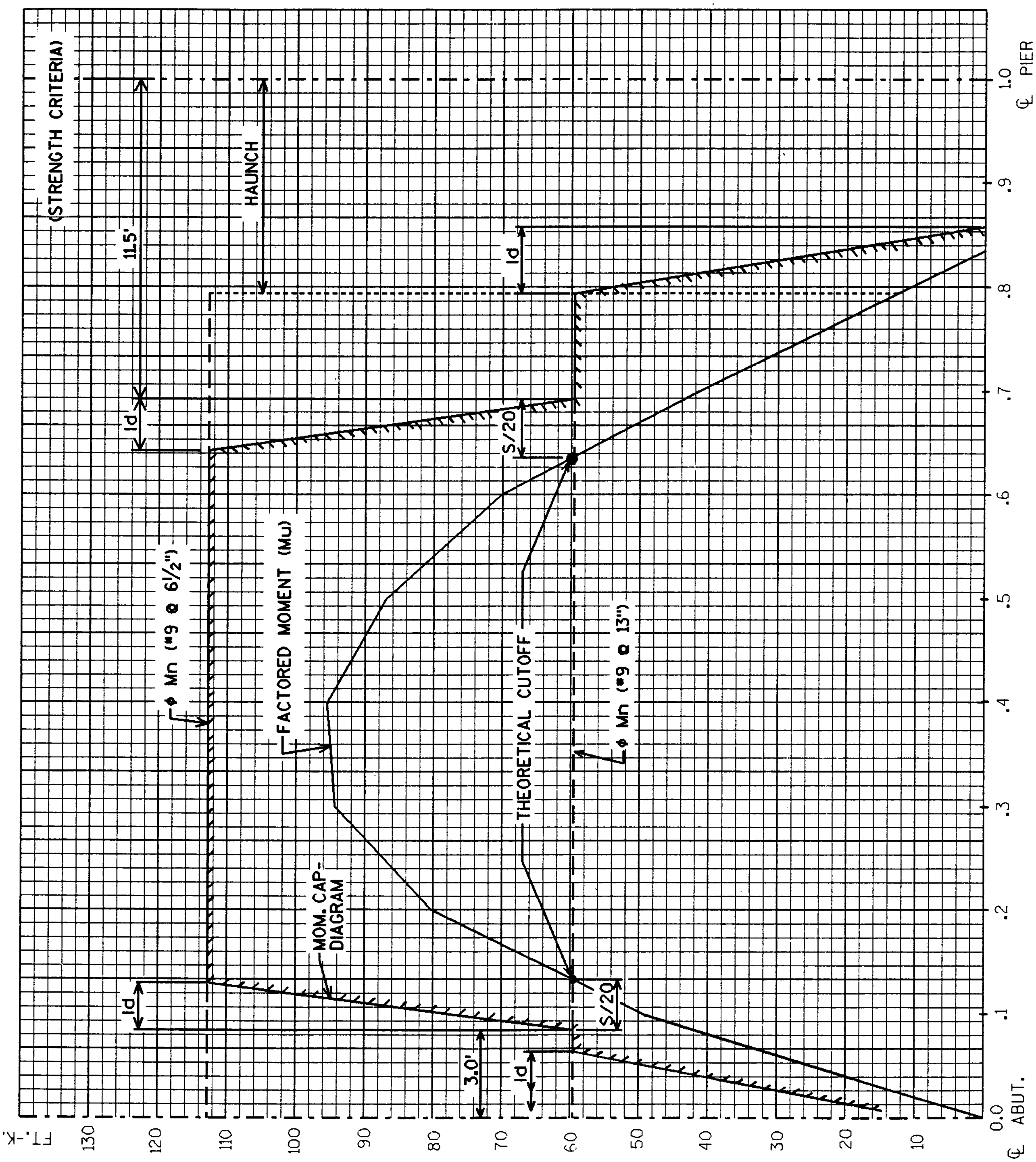
therefore, a = 2.72"

$$\phi\,Mn = \phi[A_s\,(fy)(d-\frac{a}{2})] = 0.9[(111.0)(14.9-\frac{2.72}{2})]/12 = 112.7\ ft.-k.$$

Capacity of #9 at 13" (A_s = 0.93 in^2/ft.) d = 14.9"

C = 40.8(a)

T = A_s(fy) = (0.93)(60) = 55.8$^{k.}$, C = T



SPAN (1) - POSITIVE MOMENT CUTOFF DIAGRAM therefore, a = 1.37"

$$\phi Mn = \phi\left[A_s(fy)(d-\frac{a}{2})\right] = 0.9\left[(55.8)(14.9-\frac{1.37}{2})\right]/12 = 59.5 \text{ ft-k.}$$

The moment diagram equals the capacity of #9 at 13" at 4.9' from C/L abutment. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span, whichever is greater. (AASHTO 8.24.1)

d(eff.) = 14.9" L_d(#9) (See Table 9.4, Chapter 9).
$15(d_b) = 15(1.128") = 16.9"$
S/20 = 38'/20 = <u>1.9' controls</u>

Therefore, we may cut 1/2 of bars at 3'-0 from the C/L of the abutment if fatigue and crack control criteria are satisfied. (Check at 0.08 pt.).

<u>Fatigue Check (at cutoff)</u> (0.08 pt.)

<u>Steel:</u> At 0.08 point of span 1 the moment range is [+(L+I)] - [-(L+I)], because moment range stays in tensile zone.
=(+13.2)-(-2.48) = 15.68 ft-k.

Allowable f_f = 21.-.33fmin.+(8)(.3) = 21.55 k.s.i.

where $.33f_{min.} = \dfrac{0.33\left[(DIM + Curb\ DL - (L+I))\times 12\right]}{(A_s)\ (j)\ (d)}$

$$= \frac{0.33\left[(7.12 + 1.2 - 2.48)(12)\right]}{(.93)(.9)(14.9)} = 1.85 \text{ k.s.i.}$$

$$A_s(\min) = \frac{M(range)}{f_f\ j\ d} = \frac{(15.68)(12)}{(21.55)(.9)(14.9)} = 0.65in^2/ft. < 0.93in^2/ft.\ (O.K.)$$

<u>Crack Control Check (at cutoff).</u> (0.08 pt.)

$d_c = 1.5 + (1.128)/2 = 2.064$ in.

$A = (2)(2.064)(13) = 53.66in^2$

$$f_s allow. = \frac{170}{[(2.064)(53.66)]^{1/3}} = 35.4 \text{ k.s.i.} < 0.6fy$$

$$f_s(act) = \frac{(DIM + Curb\ DL + (L+I))}{(A_s)\ (j)\ (d)} = \frac{(7.12 + 1.2 + 13.2)(12)}{(0.93)(.9)(14.9)}$$

= 20.70 k.s.i. < 35.4 k.s.i. (O.K.)

Max. reinf. check. O.K.
Min. reinf. check. O.K.

∴ Cut 1/2 of bars at 3'-0" from C_L of abutment. Remaining bars are extended into support. (AASHTO 8.24.2)

The moment diagram equals the capacity of #9 at 13" at 13.7' from C/L pier. Reinforcement shall be extended S/20 beyond this point, which will be at 0.70 pt. Therefore, we can cut 1/2 of bars at 11'-6" from C/L of pier if fatigue and crack control criteria are satisfied.

Fatigue Check (at cutoff) (0.70 pt.)

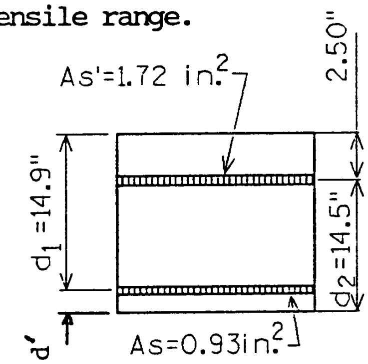
Steel: At 0.70 point of span 1 the moment range goes from tension to compression.

Tensile moment range = DLM + (+L+I) = -3.9 + 21.4 = +17.5 ft.-k

Compression moment range = DLM + (-L+I) = -3.9 -21.7 = -25.6 ft-k.

Curb DL was ignored in order to obtain a greater tensile range.

The tensile part of the stress range in the bottom bars is computed as:



$$fs = \frac{(+M)}{A_s\ j\ d_1} = \frac{17.5\ (12)}{(.93)(.9)(14.9)}$$

= 16.84 k.s.i. (T)

The compressive part of the stress range in the bottom bars is computed as:

$$f_s' = \frac{(-M)}{A_s' j\ d_2}\ \frac{(k - (d'/d_2))}{(1 - k)} = \frac{(25.6)(12)}{(1.72)(.9)(14.5)}\left[\frac{0.3-(2.064)/14.5)}{1 - 0.30}\right]$$

= 3.08 k.s.i. (C)

We have assumed (#8 at 5 1/2") req'd. at pier, is present at this location as compression steel (A_s').

Therefore, total stress range on bottom steel = $f_s + f_s'$ = 19.92 k.s.i.

Allowable f_f = 21.-.33fmin.+(8)(.3) = 24.42 k.s.i. > $f_s + f_s'$ (O.K.)

where .33fmin. = .33f's = .33(-3.08) = -1.02 k.s.i.

Crack Control Check (at cutoff). (0.70 pt.)

d_c = 1.5" + (1.128)/2 = 2.064"

A = (2)(2.064)(13) = 53.66in.^{2}

$$f_s \text{allow.} = \frac{170}{[(2.064)(53.66)]^{1/3}} = 35.4 \text{ k.s.i.} < 0.6 \text{ fy}$$

$$f_s(\text{act.}) = \frac{(DLM + (L+I))}{A_s\, j\, d} = \frac{(-3.9 + 21.4)(12)}{(.93)(.9)(14.9)}$$

$$= 16.84 \text{ k.s.i.} < 35.4 \text{ k.s.i.} \text{ (O.K.)}$$

Curb DL was ignored in order to obtain a greater tensile moment.

Max. reinf. check. O.K.

Min. reinf. check. O.K.

∴ Cut 1/2 of bars at 11'-6" from C/L of pier. Remaining bars are extended (Ld) beyond haunch-slab intercept as shown on Standard 18.1.

Span 2 Positive Moment Reinforcement. (Cutoffs)

Preliminary bar steel cutoff locations for positive moment are determined when one-half the steel required at the 0.5 pt. has the capacity to handle the ultimate moment at these locations. However, the service load stresses must meet the fatigue and crack control requirements at the cutoff locations. The factored moments (Mu) at the 0.1 points have been plotted on the following page. The capacities (ØMn) of #8 at 5 1/2" and #8 at 11" are also shown. b = 12" (for 1'-0 design width).

Capacity of #8 at 5 1/2" (A_s = 1.72 in^2/ft.) d = 15.0" (AASHTO 8.16.3.2)
ØMn = 106.3 ft.-k ,

Capacity of #8 at 11" (A_s = 0.86 in^2/ft.) d = 15.0"
ØMn = 55.6 ft.-k. ,

The moment diagram equals the capacity of #8 at 11" at 14.8' from C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span, whichever is greater. (AASHTO 8.24.1)

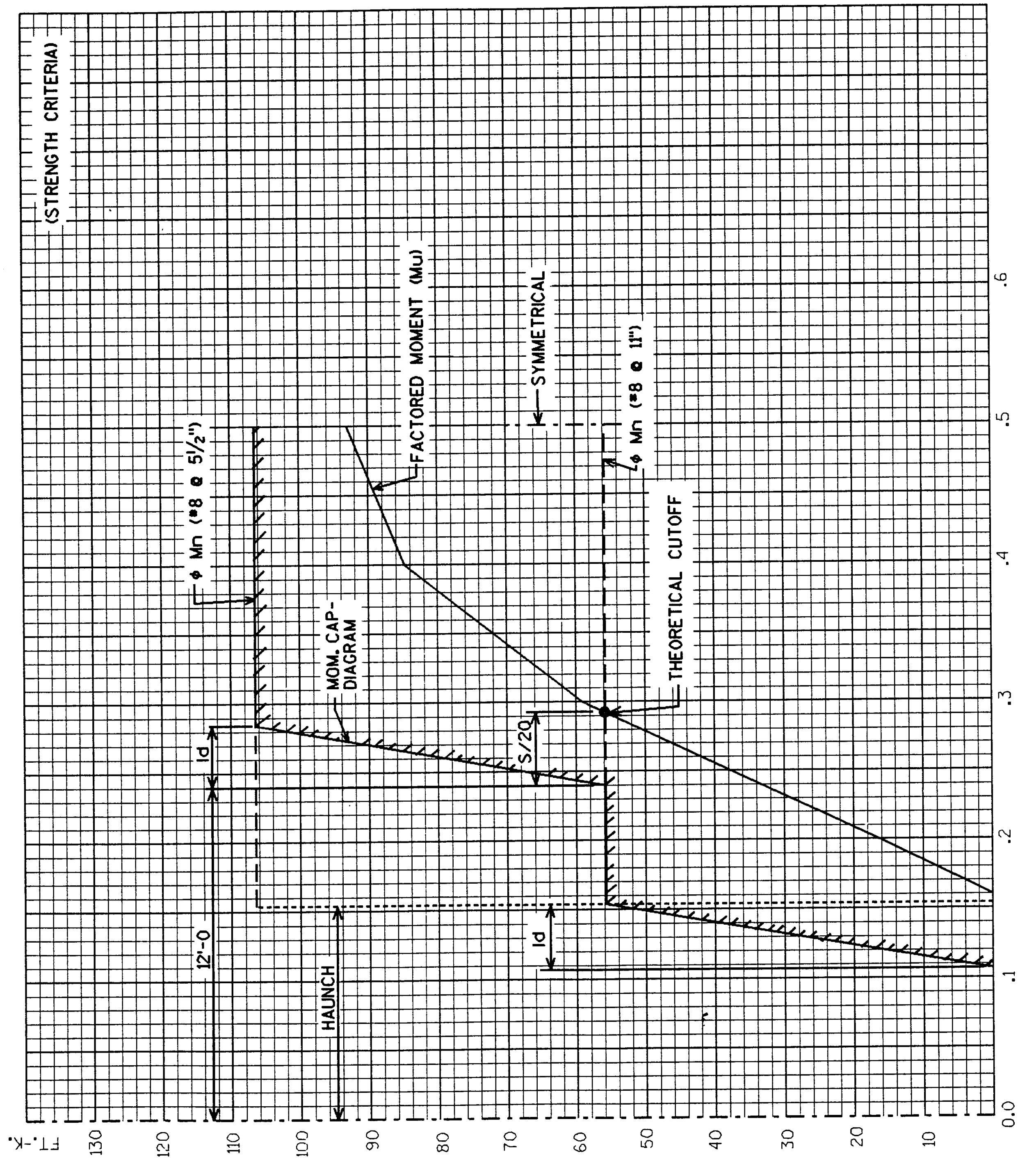
S/20 = 51'/20 = 2.55' controls. Ld (#8) (See Table 9.4, Chap. 9)

Therefore, we can cut 1/2 of bars at 12'-0" from C/L of each pier if fatigue and crack control criteria are satisfied.

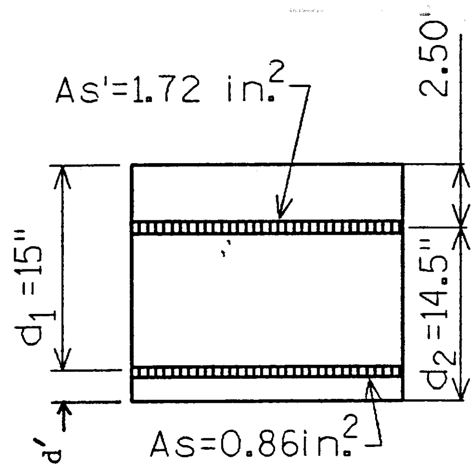
Fatigue Check (at cutoff). (0.24 pt.)

Steel: At 0.24 point of span 2 the moment range goes from tension to compression.

Tensile moment range = DLM + (+L+I) = -1.82 + 16.26 = +14.44 ft.-k.
Compressive moment range = DLM + (-L+I) = -1.82 - 16.74
= -18.56 ft.-k.



SPAN (2) - POSITIVE MOMENT CUTOFF DIAGRAM



Curb DL was ignored in order to obtain a greater tensile range.

The tensile part of the stress range in the bottom bars is computed as:

$$f_s = \frac{(+M)}{A_s\, j\, d_1} = \frac{14.44(12)}{(.86)(.9)(15.0)}$$

$$= 14.93 \text{ k.s.i. (T)}$$

The compressive part of the stress range in the bottom bars is computed as:

$$f_s' = \frac{(-M)}{A_s'\, j\, d_2}\ \frac{(k - (d'/d_2))}{1-k} = \frac{(18.56)(12)}{(1.72)(.9)(14.5)}\left(\frac{(0.3 - (2.00/14.5))}{1 - 0.30}\right)$$

$$= 2.30 \text{ k.s.i. (c)}$$

We have assumed (#8 at 5 1/2") req'd. at pier, is present at this location as compression steel (A_s').

Therefore, total stress range on bottom steel $= f_s + f_s' = 17.23$ k.s.i.

Allowable f_f = 21. − .33fmin. + (8)(.3)
= 24.2 k.s.i. > $f_s + f_s'$ (O.K.)

where .33fmin. = .33(f's) = .33 (−2.30) = −0.76 k.s.i.

Crack Control Check (at cutoff). (0.24 pt.)

d_c = 1.5" + 1.00/2 = 2.00"

A = (2)(2.00)(11") = 44.0 in^2.

$$f_s\text{allow.} = \frac{170}{[(2.00)(44.0)]^{1/3}} = 38.22 \text{ k.s.i.} > 0.6 \text{ fy}$$

therefore $f_s$allow. = 36 k.s.i.

$$f_s(\text{act.}) = \frac{(DLM + (L+I))}{A_s\ j\ d} = \frac{(-1.82 + 16.26)(12)}{(.86)(.9)(15.0)}$$

$$= 14.92 \text{ k.s.i.} < 36 \text{ k.s.i. (O.K.)}$$

Curb DL was ignored in order to obtain a greater tensile moment.

Max. reinf. check. O.K.
Min. reinf. check. O.K.

ꝏ Cut 1/2 of bars at 12'-0" from C_L of each pier. Remaining bars are extended (Ld) beyond haunch-slab intercept as shown on Standard 18.1.

Span 1. Negative Moment Reinforcement.

Preliminary bar steel cutoff location for negative moment is determined when one-half the steel required at the pier has the capacity to handle the ultimate moment at this location. However, the service load stresses must meet the fatigue and crack control requirements at the cutoff locations. The factored moments (Mu) at the 0.1 points have been plotted on the following page. The capacities (ϕMn) of #8 at 5 1/2" and #8 at 11" are also shown.

b = 12" (for a 1'-0 design width)

Capacity of #8 at 5 1/2" (A_s = 1.72 in^2/ft.) (AASHTO 8.16.3.2)

ϕMn = 187.6 ft.-k (at C/L Pier) , d = 25.5"
ϕMn = 102.4 ft.-k (in span) , d = 14.5"

Capacity of #8 at 11" (A_s = 0.86 in^2/ft.)

ϕMn = 96.2 ft.-k (at C/L pier) , d = 25.5"
ϕMn = 53.7 ft.-k (in span) , d = 14.5"

The moment diagram equals the capacity of #8 at 11" at the 0.7 pt. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span, whichever is greater. (AASHTO 8.24.1)

S/20 = 38'/20 = 1.9' controls Ld(#8) (See Table 9.4, Chap. 9)

Therefore, we can cut 1/2 of bars at 13'-6" from C/L of pier if fatigue and crack control criteria are satisfied.

Fatigue Check (at cutoff) (0.65 pt.)

Steel: At 0.65 point of span 1 the moment range goes from tension to compression.

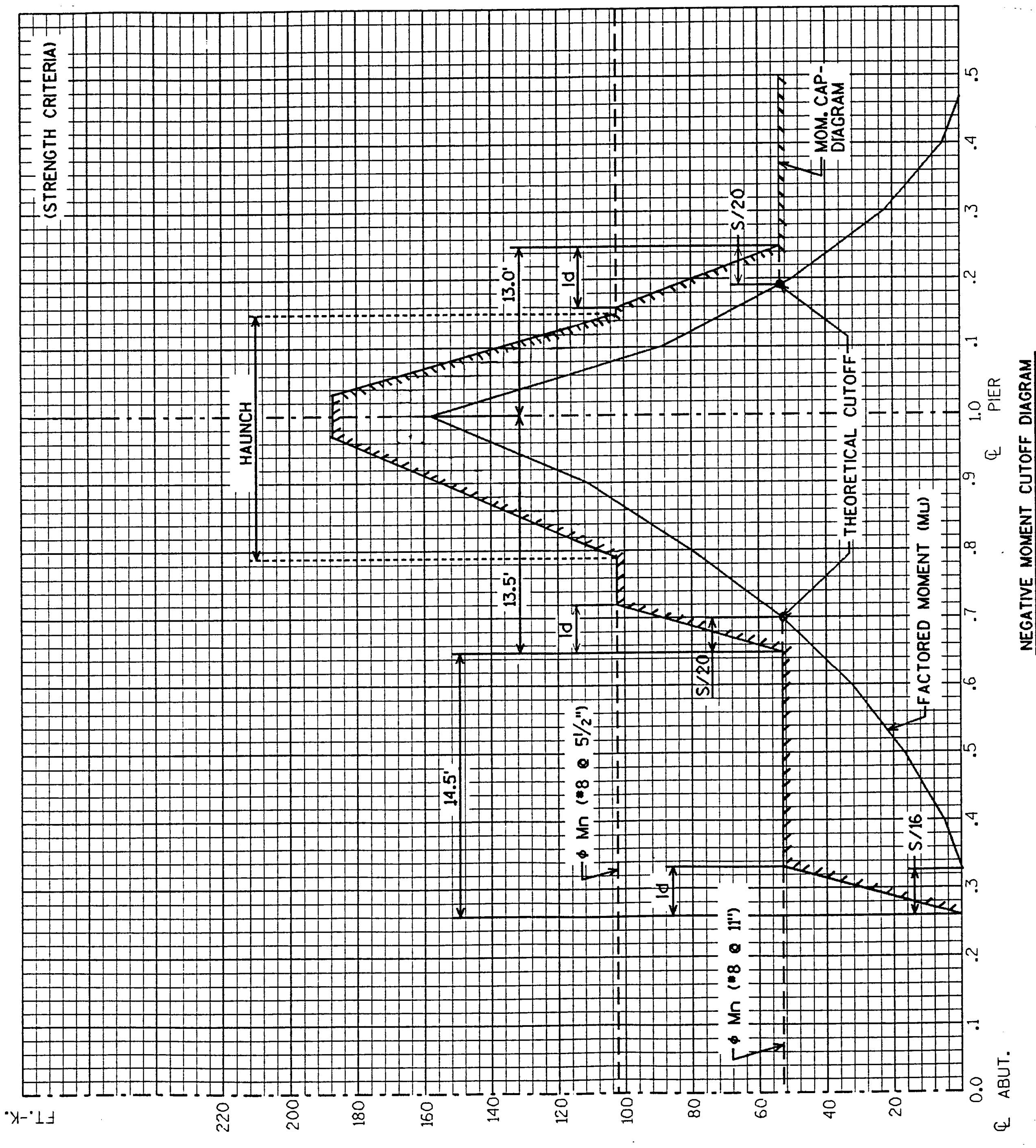
Tensile moment range = DLM + (-L+I) = 1.1 -20.15 = -19.05 ft.-k.

Compressive moment range = DLM + (+L+I) = 1.1 + 24.75 = +25.85 ft.-k.

Curb DL was ignored in order to obtain a greater tensile range.

The tensile part of the stress range in the top bars is computed as:

$$f_s = \frac{(-M)}{A_s\ j\ d_1} = \frac{19.05(12)}{(.86)(.9)(14.5)} = 20.37 \text{ k.s.i.} (T)$$


(STRENGTH CRITERIA)
NEGATIVE MOMENT CUTOFF DIAGRAM
MOM. CAP- DIAGRAM
THEORETICAL CUTOFF
FACTORED MOMENT (Mu)
HAUNCH
13.0'
13.5'
14.5'
ld
S/20
S/16
φ Mn (#8 @ 5½")
φ Mn (#8 @ 11")
FT.-K.
℄ ABUT.
℄ PIER
0.0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
20
40
60
80
100
120
140
160
180
200
220

The compressive part of the stress range in the top bars is computed as:

$$f_s' = \frac{(+M)}{A_s'\ j\ d_2}\ \frac{(k-(d'/d_2))}{1-k} = \frac{25.85(12)}{(1.85)(.9)(14.9)}\left[\frac{(0.3-(2.50/14.9))}{1-0.3}\right]$$

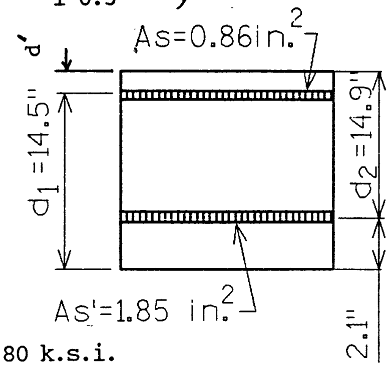
$= 2.4$ k.s.i. (C)

We have (#9 at 6 1/2") as compression steel (A_s') at this location.

Therefore, total stress range on top steel $= f_s + f_s' = 22.77$ k.s.i.

Allowable $f_f = 21. - .33fmin + 8(.3)$
$= 24.20$ k.s.i. $> f_s + f_s'$ (O.K.)

where $.33fmin. = .33(f_s') = .33\ (-2.4) = -0.80$ k.s.i.



Crack Control Check (at cutoff). (0.65 pt.)

$d_c = 2.5" - 0.5" + 1.000/2 = 2.50"$

$A = (2)(2.50")(11") = 55.0\ in^2$

$$f_s\ allow. = \frac{130}{[(2.50)(55.0)]^{1/3}} = 25.19 \text{ k.s.i.} < 0.6 \text{ fy}$$

$$f_s(act.) = \frac{(DLM+(-L+I))}{A_s\ j\ d} = \frac{(1.1-20.15)(12)}{(.86)(.9)(14.5)} = 20.37\ k.s.i.(T) < 25.19\ k.s.i.\ (O.K.)$$

Curb DL was ignored in order to obtain a greater tensile moment.

Max. reinf. check. O.K.

Min. reinf. check. O.K.

∴ Cut ½ of bars at 13'-6 from C/L of the pier. Remaining bars are extended beyond the point of inflection a distance equal to the effective depth of the member, 12 bar diameters, or 1/16 of the clear span, whichever is greater. (AASHTO 8.24.3).

d(eff) = 14.5"
$12(d_b) = 12(1.00") = 12.0"$

Ld(#8) (See Table 9.4, Chapter 9)

S/16 = 38'/16 = 2.38' controls

Looking at the factored moment diagram (Mu) on page 36, we find point of inflection at 0.33 pt. Therefore cut remaining bars at 28'-0" from C/L pier. Lap these bars with #4 at 11".

Span 2 Negative Moment Reinforcement (Cutoffs)

Preliminary bar steel cutoff location for negative moment is determined when one-half the steel required at the pier has the capacity to handle the ultimate moment at this location. However, the service load stresses must meet the fatigue and crack control requirements at the cutoff locations. The factored moments (Mu) at the 0.1 points have been plotted on page 36. b = 12" (for a 1'-0 design width)

Capacities of #8 at 5 ½" and #8 at 11" are as stated on page 35.

The moment diagram equals the capacity of #8 at 11" at the 0.20 pt. Reinforcement shall be extended beyond the point a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span, whichever is greater. (AASHTO 8.24.1)

S/20 = 51'/20 = 2.55' controls. Ld(#8) (See Table 9.4, Chapter 9).

Therefore, we can cut ½ of bars at 13'-0" from C/L of pier if fatigue and crack control criteria are satisfied.

Fatigue Check (at Cutoff) (0.25 pt.)

Steel: At 0.25 point of span 2 the moment range goes from tension to compression.

Tensile moment range = DLM + Curb DL +(-L+I) = -0.4 – 0.05 – 16.4
= 16.85 ft.-k.

Compressive moment range = DLM + Curb DL + (+L+I) = -0.4 –0.05 + 17.3
= 16.85 ft.-k.

The tensile part of the stress range in the top bars is computed as:

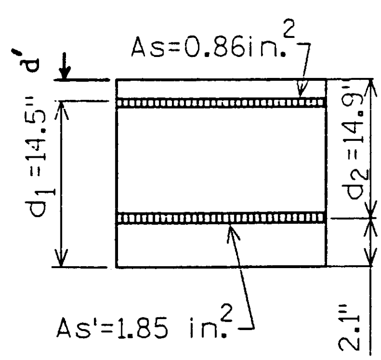
$$f_s = \frac{(-M)}{A_s\ j\ d_1} = \frac{16.85(12)}{(.86)(.9)(14.5)} = 18.02\ k.s.i.(T)$$

The compressive part of the stress range in the top bars is computed as:

$$f_s' = \frac{(+M)}{A_s'\ j\ d_2}\ \frac{(k-(d'/d_2))}{1-k}$$

$$= \frac{16.85(12)}{(1.85)(.9)(14.9)}\left[\frac{(0.3 - (2.5/14.9))}{1-0.3}\right]$$

$$= 1.54\ k.s.i \quad (C)$$



We have (#9 at 6 ½") as compression steel (A_s') at this location.

Therefore, total stress range on top steel = $f_s + f_s'$ = 19.56 k.s.i.

Allowable f_f = 21. - .33fmin + (8) (.3) = 23.91 k.s.i. > $f_s + f_s'$ (O.K.)

Where .33fmin = .33 (f_s') = .33(-1.54) = -0.51 k.s.i.

Crack Control Check (at cutoff) (0.25 pt.)

d_c = 2.5" – 0.5" + 1.000/2 = 2.50"

A = (2)(2.50")(11") = 55.0 in^2

f_s allow. = 25.19 k.s.i. < 0.6 fy

$$f_s\,(act.) = \frac{(DLM + Curb\,DL + (-L+I)}{A_s \quad j \quad d} = \frac{(-0.4-.05-16.4)(12)}{(.86)(.9)(14.5)} = 18.02\,k.s.i.(T) < 25.19\,k.s.i.\ (O.K.)$$

Max. reinf. check. O.K.

Min. reinf. check. O.K.

∴ Cut ½ of bars at 13'-0 from C/L of the pier. Remaining bars are extended beyond the point of inflection a distance equal to the effective depth of the member, 12 bar diameters, or 1/16 of the clear span, whichever is greater. (AASHTO 8.24.3).

S/16 = 51'/16 = 3.19' controls Ld (#8) (See Table 9.4, Chapter 9).

Looking at the factored moment diagram (Mu) on page 36, we find point of inflection at 0.47 pt.
Therefore run remaining bars to C/L of span 2 and lap them.

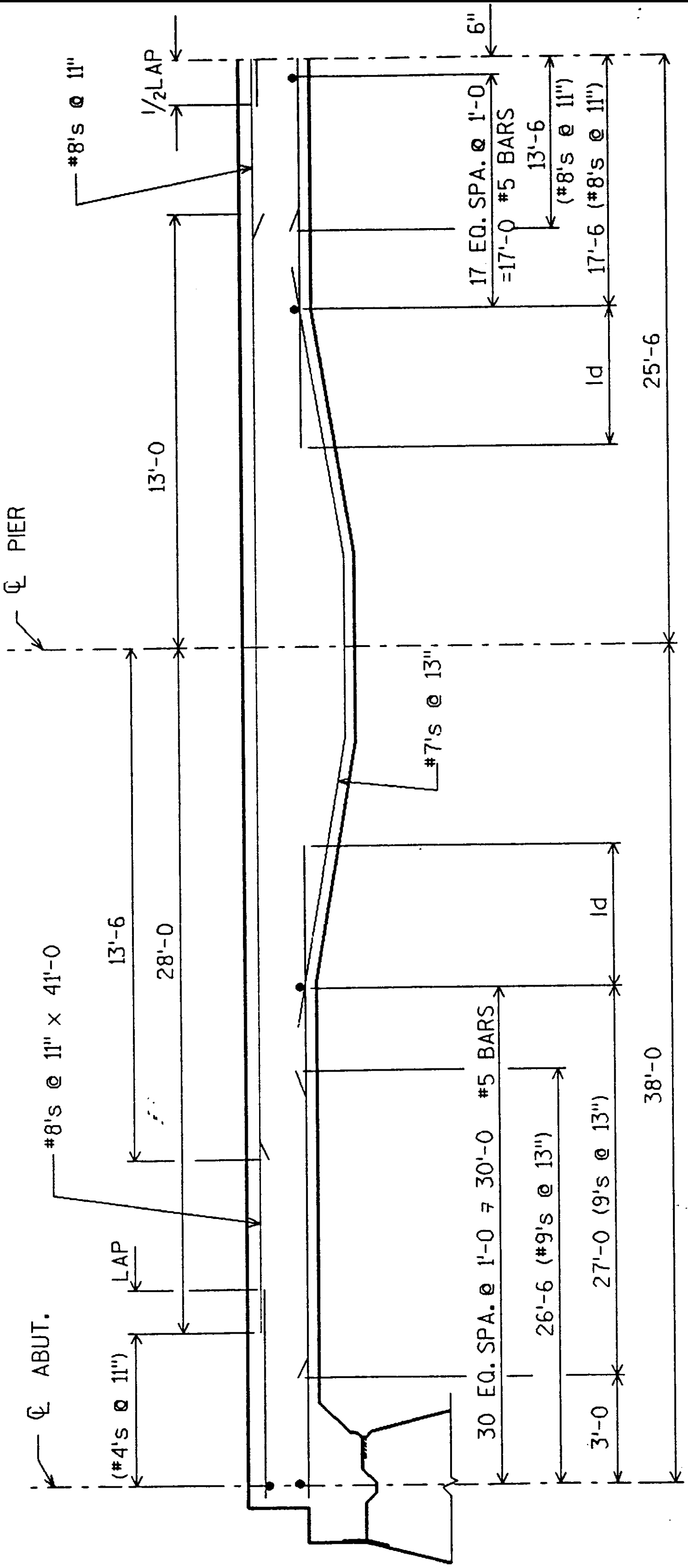
Longitudinal steel in bottom of haunch. (AASHTO 8.24.2)

At least (1/4) of positive moment reinforcement shall extend into the support.

Max. positive (A_s) = 1.85 in^2/ft.

Reinf. req'd. = (0.25) (1.85 in^2/ft.) = 0.46 in^2/ft.

∴ Use #7 at 13" (0.55 in^2/ft) > reinf. req'd. and min. reinf. on Standard 18.1 (O.K.)



SUMMARY OF LONGITUDINAL REINFORCEMENT/DISTRIBUTION STEEL

TOTAL HAUNCH THICKNESS= 2'-4½"

TOTAL SLAB THICKNESS= 1'-5½"

FIGURE 18.4

Edge Beam Design (Positive moment zone). (AASHTO 3.24.8)

Spans 1 & 3:

The slab dead load and safety parapet moments are placed on an edge beam width equal to the width of the parapet bearing on the slab (1'-1) plus one-half the depth of slab.

Edge Beam Width = $1'\text{-}1 + \frac{1'\text{-}5}{2}$ = 1'-9 1/2" = 1.79 ft.

At 0.4 pt. of span 1. (0.6 pt. of span 3).

DLM (slab + W.S.) = (1.79 ft) (16.7 ft-k/ft) = 29.9 ft-k

DLM (para) $= \frac{\text{Parapet wt./ft}}{\text{Slab wt/ft}} * (\text{DLM(slab)/ft})$

$= \frac{338}{219}(16.7) = 25.8$ ft-k

DLM (f.w.s.) $= \frac{20}{219}(1.79' - 1.08')(16.7) = 1.08$ ft-k

DLM (total) = (29.9 + 25.8 + 1.08) = 56.8 ft-k

DL M_u = 1.3 (56.8) = 73.8 ft-k

$R_n = \frac{Mu}{\phi bd^2} = \frac{(73.8)(12)(1000)}{(0.9)(21.5)(14.9)^2} = 206$ psi

$\rho = 0.00355$

$A_s = (0.00355)(12)(14.9) = 0.635\ \text{in}^2/\text{ft}$

The live load moment (incl. impact) is placed on an edge beam having a width equal to the portion overlapped by the parapet (1'-1). The depth of the section is equal to the depth of the slab plus 1'-0 of the parapet.

b = 1'-1, d = 1'-5 + 1'-0 - 2" = 2'-3

At 0.4 pt. of span 1.

LLM_t = [(LLMft-k/ft) (1/DF)] * (2 wheels) (0.2)

LLM_t = [(32.5) (1/0.159)] 0.40 = 81.8 ft-k

LLM_u = (1.3) (5/3) (81.8) = 177.2 ft-k

$R_n = \frac{(177.2)(12)(1000)}{(0.9)(13)(27)^2} = 249.3$ psi, $\rho = .0043$

A_s = (0.0043)(13)(27) = 1.52 in^2 or 1.40 in^2/ft.

A_s (total) = 0.64 + 1.40 = 2.04 in^2/ft (req'd).

A_s (prov'd) = 1.85 in^2/ft. (#9 at 6 1/2") < 2.04 in^2/ft.

Use #9 bars at 5 1/2 inch centers (A_s = 2.18 in^2/ft) in edge beam.

Span 2:

In a similar manner to Span 1, the bar steel required for the edge beam in Span 2 is computed at 0.5 pt. of span 2.

A_s (DLM) = 0.69 in^2./ft.

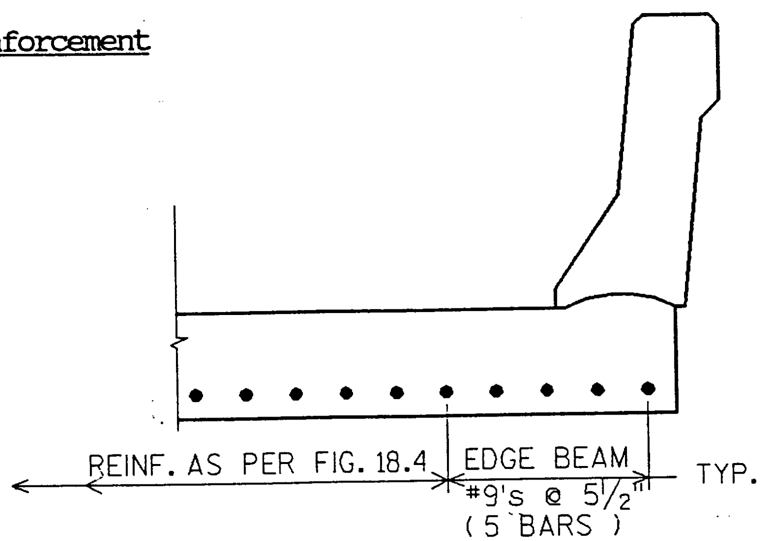
A_s (LLM) = 1.45 in^2./ft.

Total A_s = 2.14 in^2./ft.

A_s (prov'd) = 1.72 in^2./ft. (#8 at 5 1/2") < 2.14 in^2./ft.)

Use #9 bars at 5 1/2 inch centers (A_s = 2.18 in^2/ft.) in edge beam.

Edge Beam Reinforcement



Note: Place deflection joints equidistant on either side of point of maximum positive moment. See Section 18.4(2) in this chapter and Standard 18.1 for additional information on proper location of deflection joints.

Distribution Reinforcement (Transverse bottom reinf.). (AASHTO 3.24.10)

The criteria for main reinforcement parallel to traffic is applied; the percentage of distribution reinforcement is based on a fraction of the positive moment steel.

Spans 1 & 3:

Percentage = $\frac{100}{\sqrt{S}}$ x 100% ≤ 50% MAX, where S is the span length in feet. Main positive reinforcement equals #9's @ 6 1/2" ctrs.

Percentage = $\frac{100}{\sqrt{38}}$ x 100% = 16.2% ≤ 50% MAX

A_s = (0.162)(1.85) = 0.30 in^2/ft

= #5's @ 1'-0" ctrs.

Span 2:

Percentage = $\frac{100}{\sqrt{51}}$ x 100% = 14.0% ≤ 50% MAX.

A_s = (0.140)(1.72) = 0.24 in^2/ft

= #5's @ 1'-0" ctrs.

Refer to Standard 18.1 for placement of distribution reinforcement. For simplicity, we have placed distribution reinforcement as per Figure 18.4.

Note: Shear and Bond (O.K.) per (AASHTO 3.24.4).

Transverse Reinforcement at Piers (Haunch w/o transverse taper).

Cap-type pier with rounded cap ends, used in this example.

Transverse Haunch Length:

Out-to-Out of Slab = 42'-2"

Length of haunch along skew = 42'-2"/cos 6^o
= 42.40 ft.

Length of Pier cap = 42.40' - (2) [(1'-3" + 6")/cos 6^o - (1.25')] = 41.38'

Assume (4) columns are to be used and pier cap is 2'-6" X 2'-6"

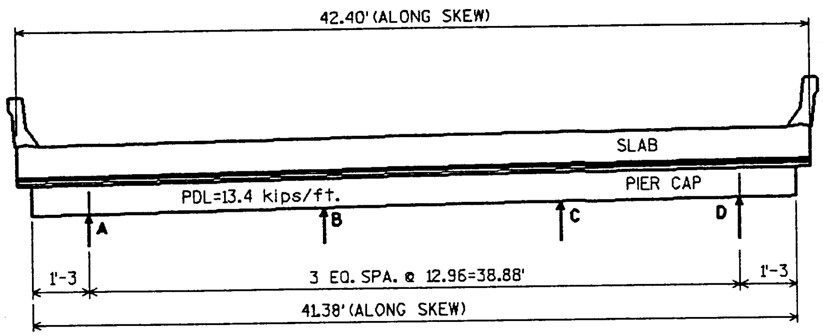
Dead Load Moment (on pier cap).

From the computer analysis, S_{DL} = 12.4 k/ft. at the piers. (DL shear span 1 = 6.2^k and DL shear span 2 = 6.2^k, on a 1'-0 slab width).

S_{DL} = Slab DL + 1/2" W.S., all of which is carried by pier cap.

Pier cap DL = 1.0k/ft.

$\therefore$ PDL (on pier cap) = 13.4^k/ft.



$M_A = 1/2\ WL^2 = (1/2)(13.4)(1.25)^2 = 10.5$ ft.-kips $= M_D$

Applying the three-moment equation for M_B gives values of

$\frac{6A\bar{a}}{L} = \frac{6A\bar{b}}{L} = \frac{WL^3}{4}$. Other methods such as influence tables or moment distribution can also be used to obtain the dead load moments.

$$\frac{WL^3}{4} = \frac{(13.4)(12.96)^3}{4} = 7292 \text{ ft-k}$$

The three-moment equation is

$$M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 + 6\frac{A1\bar{a}1}{L1} + 6\frac{A2\bar{b}2}{L2} = 0$$

Refer to "Strength of Materials" by Singer pages 270-277[4] for derivation of the three-moment equation.

If M_A is known and due to symmetry $M_B = M_C$; the above equation reduces to one unknown, M_B, as follows:

$$(-10.5)(12.96) + (2)(M_B)(12.96 + 12.96) + (M_B)(12.96) + 7292. + 7292. = 0$$

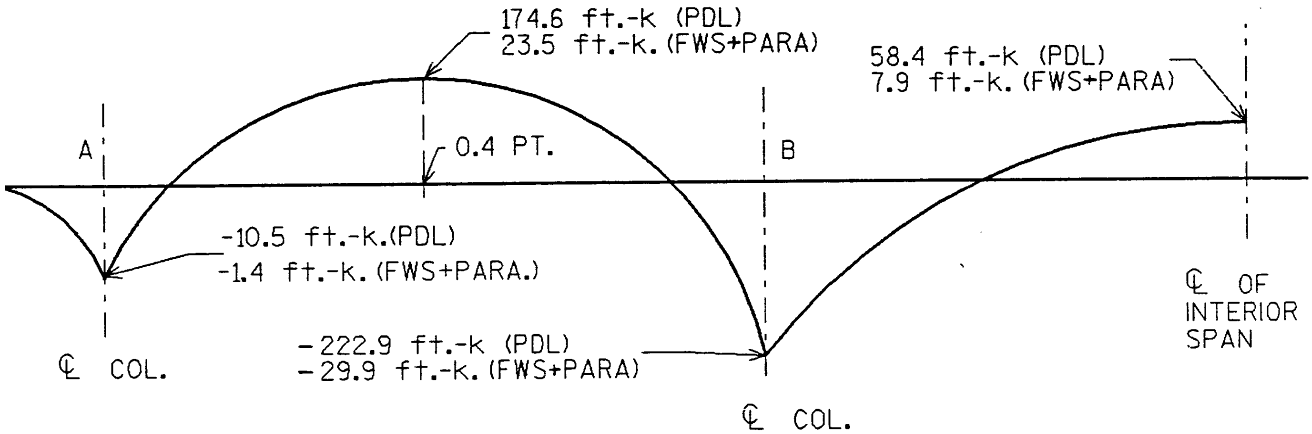
$$M_B = 222.9 \text{ ft-kips} = M_C$$

FWS + para. (DL) = 1.8^k/ft. at the piers, (on a 1'-0 slab width).

This load is carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

Using three-moment equation, $M_B = M_C = 29.9$ ft-kips.
$M_A = M_D = 1.41$ ft-kips.

The partial moment diagram for PDL and FWS + para (DL) is as follows:



Live Load Moments

Pier Lane = 61.1 k/lane
Pier Truck = 68.1 k/truck

Impact $= \dfrac{50}{12.96+125} = 36.4\%$; Use: 30% MAX. (AASHTO 3.8.2)

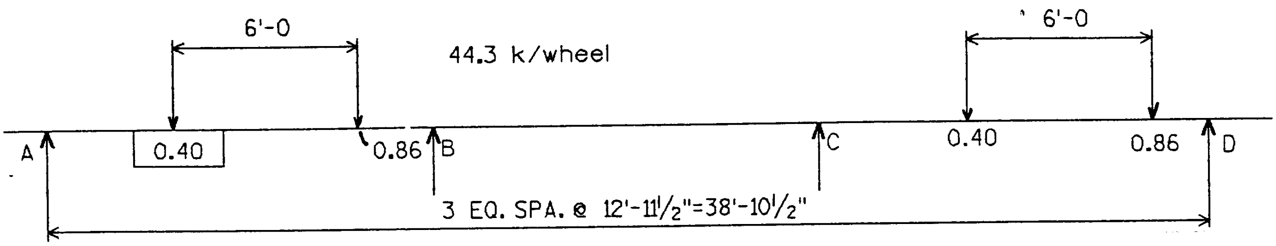
Max. Wheel Load $= \dfrac{(68.1)}{2}(1.30)$

$= 44.3$ k/wheel

This load is carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

Using influence lines for a 3-span continuous beam; the following results are obtained.

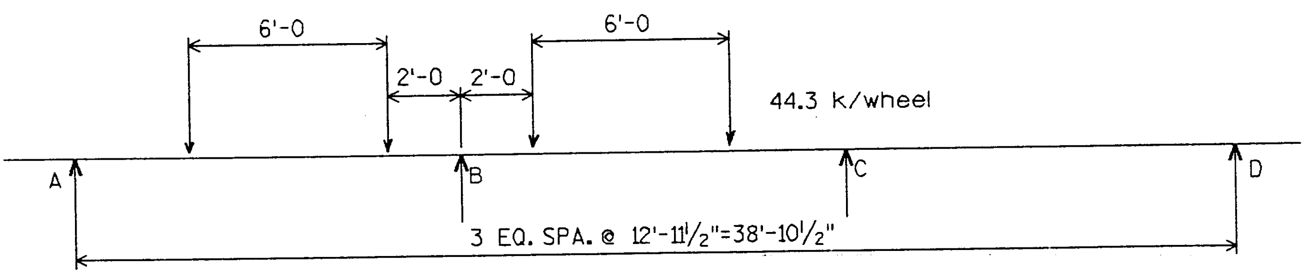
+ LLM at 0.4 Point of Exterior Span



+ LLM @ 0.4 Pt. = (0.2042+0.0328+0.0102+0.0036) (44.3) (12.96)

= 144.0 ft-k (Max + LLM Ext. Span)

− LLM at Center Line of Support



− LLM @ Support = (0.0862+0.0612+0.0515+0.0610) (44.3) (12.96)

= 149.2 ft-k (Max − LLM, Over Column)

We assumed for this example that adequate shear transfer has been achieved (AASHTO 8.16.6.5) between transverse slab member and pier cap and that they will perform as a unit. Therefore FWS + para (DL) and LLM will be acting on a member made up of the pier cap and the transverse slab member. Designer must insure adequate transfer before proceeding in this manner.

Positive moment region. (for pier cap - slab unit)

Width of slab section = 1/2 center to center column spacing or 8 feet, whichever is smaller.

(C/L-C/L) column spacing x (1/2) = 6.5' < 8.0'

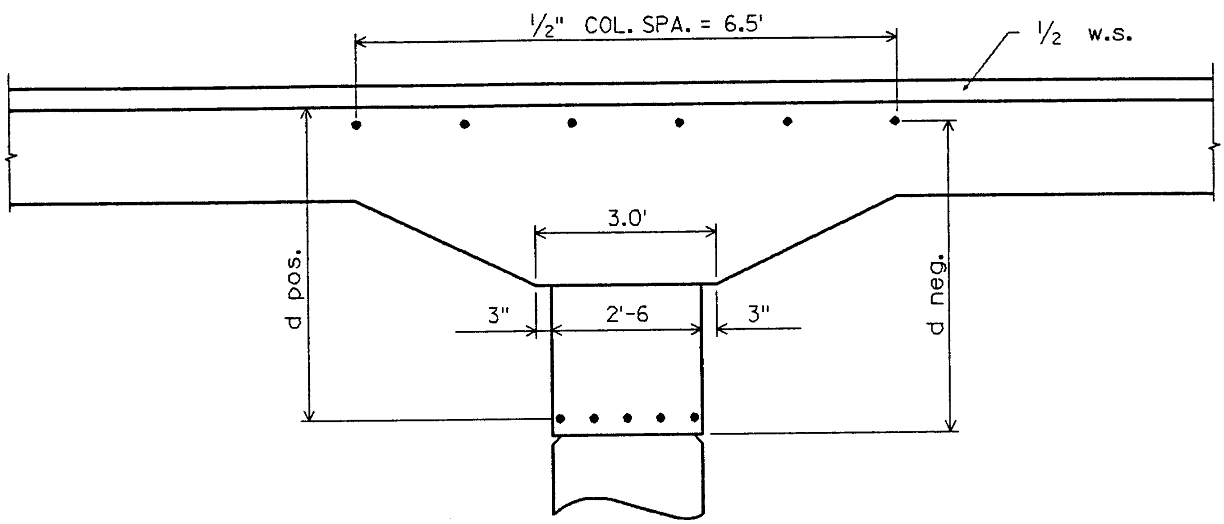
b = 6.5' = 78"

d = 28" + 30" − 1.5" − 0.625" − 0.4" = 55.5"

Negative moment region. (for pier cap - slab unit)

b = width of cap = 2.5' = 30"

d = 28" + 30" - 2" - 1" - 0.4" = 54.6"



Postitive reinforcement for Pier Cap. (0.4 Pt).

All Slab DL + 1/2" W.S. + Cap DL, (PDL), is carried by the pier cap.

b = 2.5' = 30" (pier cap)

d = 30" - 1.5" - 0.625" - 0.4" = 27.5" (pier cap)

M_u = 1.3 (PDL) = 1.3(174.6^{1-k}) = 227ft-k.

$R_n = \frac{Mu}{\phi bd^2} = \frac{(227)(12)(1000)}{(0.9)(30)(27.5)^2}$ = 133 p.s.i.

ρ = 0.00228

A_{s1} = 1.88 in^2

FWS + para (DL) and LLM will be carried by pier cap and the transverse slab member.

b = 78”

d = 55.5”

Mu = 1.3 [(DL) + 5/3 (LLM)] = 1.3 [23.5 + (5/3)(144.0)] = 342.6 ft-k.

$$R_n = \frac{Mu}{\phi bd^2} = \frac{(342.6)(12)(1000)}{(0.9)(78)(55.5)^2} = 19.0\ p.s.i$$

ρ = 0.00032

A_{s2} = 1.39 in ^{2}

A_s (total) = A_{s1} + A_{s2} = 1.88 + 1.39 = 3.27 in^2

Negative reinforcement for Pier Cap (column ‘B’).

All Slab DL + ½” W.S. + Cap (DL), (PDL), is carried by the Pier Cap.

b = 30” (pier cap)

d = 27.5” (pier cap)

Mu = 1.3 (PDL) = 1.3 (222.9) = 289.8 ft-k.

$$R_n = \frac{Mu}{\phi bd^2} = \frac{(289.8)(12)(1000)}{(0.9)(30)(27.5)^2} = 170.3 \text{ p.s.i.}$$

$\rho = 0.00294$

A_s = 2.42 in^2

Positive reinforcement for Transverse Slab Member.

See Standard 18.1 for minimum reinforcement at this location.

Negative reinforcement for Transverse Slab Member (column ‘B’).

FWS + para (DL) and LLM will be carried by pier cap and the transverse slab member.

b = 30”
d = 54.6”
Mu = 1.3 [(DL) + 5/3 (LLM)] = 1.3 [(29.9) + 5/3 (149.2)] = 362.1 ft-k.

$$R_n = \frac{Mu}{\phi bd^2} = \frac{(362.1)(12)(1000)}{(.9)(30)(54.6)^2} = 54.0\ p.s.i.$$

$\rho = 0.0009$

$A_s = 1.47\ in^2$

Shear Crack of Transverse Slab Member. (AASHTO 8.16.6.6)

Due to the geometry and loading, stirrups are generally not required or recommended.

Two-way action:

FWS + para (DL) = DLV = $(1.8^{k/ft.})(42.4') = 76.32^k$

LLV = $(44.3^k$/wheel) (6 wheels)(90% red. fact.) = 239.22^k

Vu = 1.3 (DLV + 5/3 (LLV)) = 617.5^k

$$Vu = \frac{Vu}{\phi(b_o d)} = \frac{617.5^k\ (1000)}{(0.85)(94.5')(12)(24.6")} = 27\ p.s.i. < 4\sqrt{f'c}$$

$\therefore$ No Stirrups are required.

Note: Shear check and shear reinforcement design for pier cap is not shown in this example. Also crack control, max. (ρ) and min (ρ) checks are not shown for pier cap.

Max. reinf. check. (AASHTO 8.16.3.1) Transverse Slab Member.

max. (ρ) = 0.75 (ρ_b) = 0.0214 (not incl. ρ')

actual $((\rho) = A_s/(b)(d) = (1.47)/(30)(54.6) = 0.0009 \langle \max.(\rho)\ O.K.$

Min. reinf. check. (AASHTO 8.17.1) Transverse Slab Member.

$\phi Mn(A_s = 1.47\ in^2) = 358.3\ ft - k < 1.2\ Mcr$

Use 4/3*(A_s req'd. for strength) = (4/3) (1.47in^2) = 1.96 in^2

1.96in^2/6.5' = 0.30in^2/ft., Try #5 at 1'-0 for 6'-6" transverse width over pier

Crack Control Check. Transverse Slab Member. (at interior column)

Negative moment reinforcement.

Crack control is governed by the equation:

f_s allow. = $\dfrac{z}{(dc\,A)^{1/3}}$ not to exceed 0.6 fy

z = 130 k/in. for top steel reinf.

A_s (req'd) = 1.96 in^2 (from page 49), #5 at 1'-0" spac.

d_c = 2 ½" – ½" + 1" + 0.3125" = 3.3125 in.

A = (2)(3.3125")(12") = 79.5in^2

f_s allow. = $\dfrac{130}{[(3.3125)(79.5)]^{1/3}} = 20.3\,k.s.i. < 0.6\,fy$

f_s act. = $\dfrac{(DLM + LLM)}{A_s\ j\ d} = \dfrac{(29.9 + 149.2)(12)}{(1.96)(.9)(54.6)} = 22.3\ k.s.i. > f_s\ allow.\ (N.G.)$

Try #5 at 11", (A_s = 2.14in^2).

d_c = 3.3125"

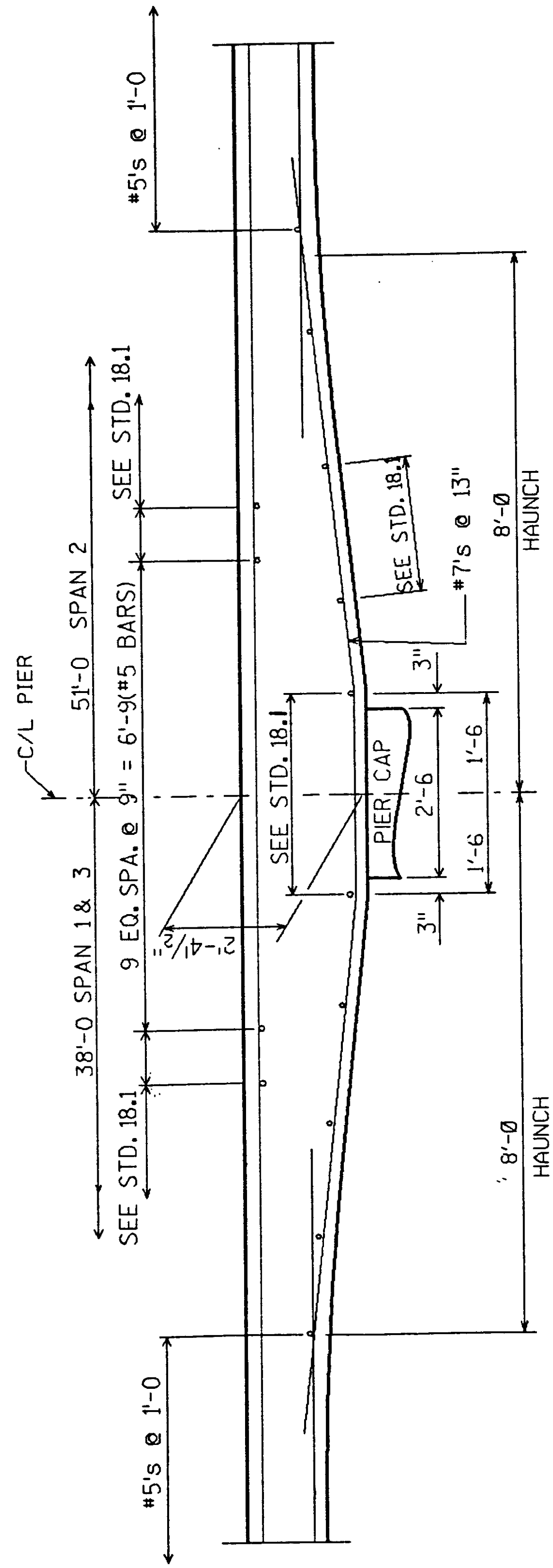
A = (2)(3.3125")(11") = 72.88in^2

f_s allow. = 20.9 k.s.i. < 0.6 fy

f_s act. = $\dfrac{(29.9 + 149.2)(12)}{(2.14)(.9)(54.6)} = 20.4\ k.s.i. < fs\ allow.\ \ (O.K.)$

Min. reinf. per Standard 18.1 is #5 at 9".

Use #5 at 9" for 6'-6" transverse width over pier.



All transverse bar steel is to be placed along skew. All transverse bars are 42'-0 long.

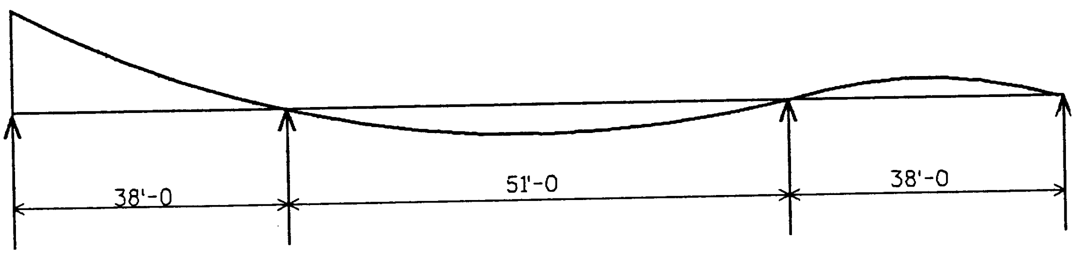
HAUNCH DETAIL

FIGURE 18.5

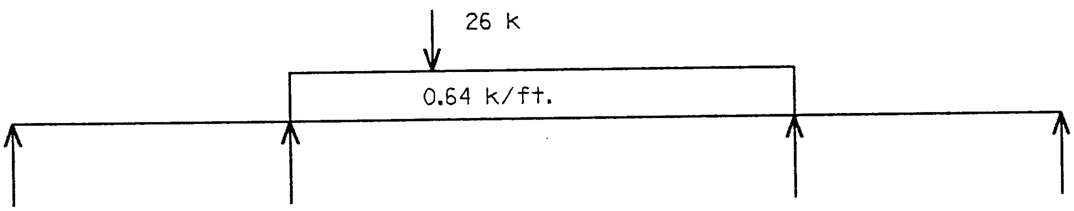
<u>Check for Uplift at Abutments</u> (AASHTO 3.17)

Maximum live load uplift at the abutments is obtained from the following influence line and loadings.

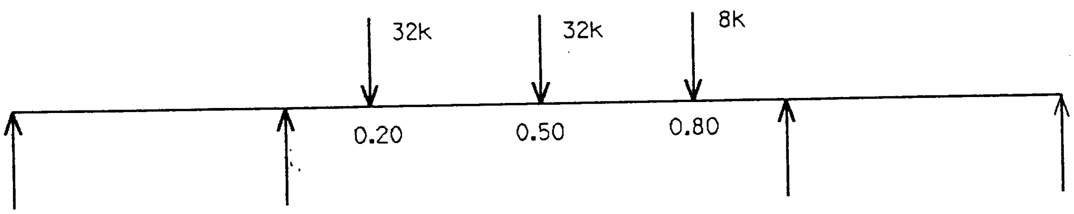
<u>INFLUENCE LINE</u>



<u>LANE LOADING</u>



<u>TRUCK LOADING</u>



Lane Load = (0.1107) (0.64) (38.0) + (0.1276) (26) = 6.0^k

Truck Load = (0.1035) + .1185) (32) + (0.0482) (8) = <u>7.5^k (Controls)</u>

$$\text{Impact} = 1 + \left[\frac{50}{51 + 125}\right] = 1.284$$

With a Safety Factor of 2;

Total Uplift = (7.5^k) (1.284) (3 Trucks) (0.90) (2.0) = 52.0^k
Dead Load at Abutments = (42.167 ft.) (2.8 k/ft. + 0.3 k/ft.) = 130.7^k > 52.0^k

Since no uplift results at the abutments, the existing dowels (#4 at 1'-0" spa.) are adequate (Standard 12.1).

Note: See Standard 18.1 for required notes and other details.

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3. Notes on ACI 318-71 Building Code Requirements with Design Applications, Portland Cement Association, "Strength Requirements", 1972, pp. 3-1 to 3-8.

4. Singer, F. L., Strength of Materials, New York, N.Y., Harper and Row, Publishers, 1962.

5. Wang, C. K., and Salmon, C. G., Reinforced Concrete Design, Scranton, Penn., International Textbook Company, 1973.

6. Notes on Load Factor Design for Reinforced Concrete Bridge Structures with Design Applications, Portland Cement Association, 1974.

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